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CONSOLIDATION OF
URANIUM MILL TAILINGS

FINAL REPORT - PART A

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Lavalin

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PART A - STATE OF THE KNOWLEDGE

SUMMARY

After reviewing the nature of the consolidation of uranium tailings in the Canadian setting, and the theories and modelling techniques for predicting the progress of consolidation in the long term, it was concluded that a computer program called PCDDF published by Cargill (1985) is the most suitable tool currently available for application in this study. The program uses the non-linear finite strain theory in analysing the consolidation of loose sediments under fully saturated condition and is capable of handling time dependent loading conditions due to accretion of tailings during deposition and lowering of the water level after completion of deposition. The effects of desiccation are dealt with by an empirical water balance approach, taking into account input by infiltration of surface water and supply of water from below due to consolidation of the saturated tailings, and loss from the tailings surface by evaporation.

An initial investigation in which the results of modelling analyses using this method were compared to field observations, established the validity of the methodology and showed that it gives a much better simulation of the progress of consolidation for uranium tailings deposited hydraulically, than the infinitesimal strain theory. The results of this initial investigation also verified the findings of other workers in this area, which indicated that the rates of consolidation of loosely deposited sediments during the progress of deposition are generally much greater than previously conceived to be, as suggested by inappropriate application of infinitesimal strain theories under such conditions. Consequently, post-deposition settlements due to consolidation under self-weight are not quite as significant as those predicted by the infinitesimal strain theory. Desiccation due to evaporation on the other hand, was found to be the most prominent process responsible for large long term settlements.

An examination of available information on existing uranium tailings deposits in Canada revealed a general deficiency in the area of material properties related to large strain consolidation behaviour and desiccation due to evaporation. Because of the sensitivity of long term settlements to the effects of desiccation, lack of information in this area made it impossible to obtain definitive predictions on long term consolidation behaviour of the three types of tailings produced in the Canadian uranium mining industry. However, through sensitivity analyses using upper and lower bound parametric values and extreme conditions, ranges of predicted long term settlements of the tailings surface with or without a thin earth/rip-rap cover over it, were obtained for three tupical cases examined. This led to the identification of the essential factors controlling long term consolidation behaviour of the tailings and the development of a decommissioning strategy which is simple and cost-effective.

The method takes full advantage of the natural shape of the tailings surface developed during deposition to enable initial drainage of the clarification pond and subsequent removal of surface run off from precipitation. This provides the necessary condition to maximize the benefit of evaporative desiccation of the most compressible upper part of the tailings, thereby removing a large portion of the long term settlement of the cover by deferring its construction for a suitable period. Strategic placement of a surcharge fill to pre-induce settlement prior to completion of the cover to further minimize differential settlement in the long term is also included as part of the scheme to ensure the integrity of the cover and the surface drainage system.

Other methods for promoting consolidation of the tailings either by predewatering prior to deposition or installation of internal drainage systems to accelerate removal of excess pore-water are also described in the report. However, the cost-effectiveness and performance over time of most of these systems are considered suspect.

Finally areas of deficiency of the current state-of-the-knowledge were identified for consideration in future efforts of research and development.

GEOCON INC.

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June 1987

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1.0 INTRODUCTION

Until very recently, uranium mill tailings have been traditionally discharged in the form of thin slurries into impoundment areas where the solid particles settle out to form soil-like deposits. The process is analogous to the formation of fluvio-lacustrine deposits in nature, with similar associated phenomena of particle sorting along the path of transport and final sedimentation of the finer silt and clay sized particles in quiescent water. The tailings deposited at the end of the sedimentation process is characterized by a very loose, metastable particle structure which subsequently undergoes large deformations under stresses imposed by accretion of tailings on top and superimposed loadings after completion of tailings deposition. Current decommissioning practices trend toward the utilization of surface drainage measures to minimize long term infiltration of precipitation into the tailings deposit and some form of top cover to mitigate radon and particulate emissions. In connection with such decommissioning practices, it becomes necessary to gain better understanding of the processes governing the consolidation behaviour of uranium tailings deposits and to develop practical methodologies for predicting their long term surface settlements.

To examine this problem in the Canadian setting, a two part study was commissioned by the National Uranium Tailings Program under the auspices of CANMET, through a Supply and Services Canada contract (No. 15SQ.23241-5-1708). This report contains the results of Part A of the study, which involves an overall review of the current state of the knowledge in connection with theories and mathematical modelling methodologies applicable to the consolidation of uranium tailings. Examination of the consolidation behaviour of three typical Canadian uranium tailings deposits, to the extent possible based on existing information, is also carried out. The stipulated Statement of Work for the study is appended to this report.

2.0 CONSOLIDATION OF URANIUM MILL TAILINGS - THE NATURE OF THE PROBLEM

The process of consolidation, as described originally by Terzaghi (1942), is a time dependent phenomenon associated with the deformation of a saturated fine-grained soil under applied loading. Because the solid particles and the pore-water are essentially incompressible, such deformations take place as a result of outflow of the pore-water, thereby allowing adjustments of the soil skeleton accompanied by reductions in the volume of the pore spaces (normally described in terms of void ratios). The impedance to flow of water through the pores of fine-grained soils results in the time dependent nature of the phenomenon.

Recognizing the basic nature of the process of consolidation, the two components of a consolidation problem can be identified to be:

- the ultimate deformation as a function of the applied load and the compressibility of the soil, and

- the rate of deformation as a function of the permeability (hydraulic conductivity) of the soil and the boundary conditions which govern the flow of the pore-water.

A study of a consolidation problem therefore involves the examination of the compressibility and permeability characteristics of the soil, together with the pertinent loading and boundary conditions.

The original formulation of the consolidation theory by Terzaghi based on certain simplifying assumptions, has led to understanding of the basic behaviour of soils and has been recognized as the foundation of modern day soil mechanics. Recent advances in the state-of-the-art have resulted in refinements of the Terzaghi theory and enabled more rigorous solutions to be obtained. Extensions of the theory to cover the consolidation of unsaturated soils have also been advanced with varying degrees of success. These theories are described in a later section in this report.

Notwithstanding major strides taken in recent years in the mathematical treatment of the problem, accurate modelling of the consolidation of tailings deposits remains difficult due to complex spatial variations of properties resulting from the manner of deposition and influence from other thermodynamic and physico-chemical processes. Bridging the gap between theory and practice in this case requires simplifying assumptions to be made which limit the degree of accuracy achievable in modelling analyses. Awareness of all the processes involved and their impact on the process of consolidation of uranium tailings, is essential in establishing the context of such analyses. It is appropriate therefore, to examine some of these influencing factors and processes at the outset of this study as discussed in the following subsections.

2.1 Geometric Aspects and Boundary Conditions

2.1.1 Impoundment Basin

Most of the existing tailings deposits are contained in basins formed by natural topographic highs combined with man-made dams. The horizontal dimensions are usually large compared to the depth of tailings in place, in ratios greater than 10:1 in most cases.

In present day practice, primary consideration is given in site selection and design of the containment dams to meet stringent targets in minimizing seepage from the tailings deposit into the surrounding natural groundwater system. Although most of the existing tailing impoundment facilities were commissioned in days past when less stringent environmental control regulations were in force, they were also designed with the intent of minimizing seepage away from the tailings.

Lateral confinement of the deposited tailings by relatively unyielding natural ground and man-made structures and the above geometric aspects and boundary drainage conditions combine to provide a setting favourable to one-dimensional consolidation of the deposited tailings with dissipation of excess pore-water pressures occurring predominantly in the vertically upward direction.

2.1.2 Top Cover and Atmospheric Environment

To meet present day requirements, a decommissioning study of a uranium tailings deposit usually includes the consideration of a top cover to mitigate radon and particulate emissions and to minimize infiltration of precipitation into the tailings deposit in the long term. Acting as a barrier between the tailings and the atmospheric environment, the top cover also has a moderating effect on the rate of exfiltration due to evaporation. The design of the top cover therefore largely controls the long term water budget of the tailings deposit and thus the ultimate position of the water level within the tailings deposit.

As discussed later, lowering of the water level results in two important effects on long term surface settlements:

1) large shrinkages of the material above the water level due to desiccation, and

2) increases in the stresses on the material below the water level accompanied by additional consolidation settlements.

It is evident therefore, that a study of post-decommissioning consolidation of a tailings deposit must take into consideration the design of the top cover. On the other hand, in designing the top cover, it is necessary to rely on predicted post-construction settlements to ensure the effectiveness of surface drainage measures. Consequently, an iterative process is involved in such problems. Taking into consideration the need in any decommissioning study to examine a number of alternative designs in connection with environmental protection, economic and practical construction aspects, it can be seen that the task of predicting the post-decommissioning consolidation behaviour of a tailings deposit is indeed not a simple matter.

2.2 Depositional Characteristics

The process of deposition of tailings in the impoundment area may extend over a period of some ten years or more at essentially continuous and constant rates. Depending on the rate of discharge, the tailings surface may rise at a rate in the order of 1 to 3 m per year. The actual rate of increase of the depth of tailings in the impoundment area has an important influence on the state of consolidation during the progress of deposition and therefore, is an important parameter in the study of the consolidation behaviour of the tailings deposit.

Grainsize distribution and void ratio are the two principal parameters which determine the permeability and compressibility of a soil. In a particular milling process, the tailings produced normally have a characteristic grind which falls within a narrow range of grainsize distributions. Minor variations within this range occur mainly as a result of changes in ore mineralogy. Due to the sensitivity of permeability to grainsize characteristics, variations in permeability of the tailings could occur as a result of such changes in the tailings grind. However, by far the most significant influence on the permeability and compressibility of the tailings is imposed by the manner of deposition in the impoundment area, as described below.

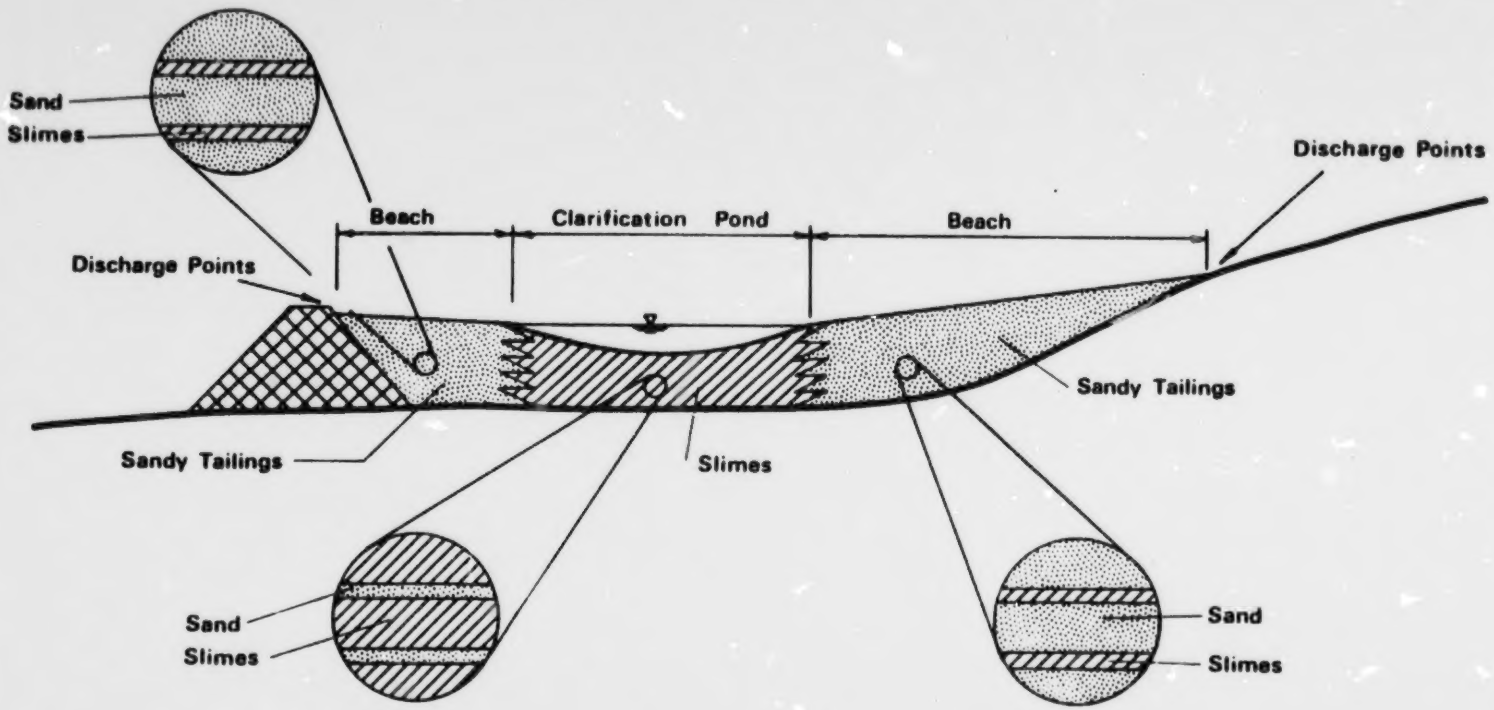
2.2.1 Slurried Tailings Deposited Hydraulically

Hydraulic deposition of tailings in thin slurries from one or more points along the perimeter of an impoundment basin, is still by far the more common practice in the Canadian uranium industry. Particle sorting in the process of sedimentation, combined with meandering flow of the ephemeral stream of tailings slurry over the surface of the impoundment area and in some cases, periodic changes in the location of discharge points, results in two principal forms of segregation within a tailings deposit:

- near horizontal stratification consisting of alternate coarse and fine grained layers
- areal segregation with coarser sandy tailings in beach areas near the discharge points and fine silty tailings (slimes) in the clarification pond area.

Such a pattern of segregation is shown schematically in Figure 2-1 on the following page. Recognition of the impact of segregation on the consolidation behaviour of the tailings deposit is of the greatest importance in predictive modelling studies. A clear understanding of the simplifications required to reduce the complex field conditions to a form suitable for input in such studies is essential for interpreting the analytical results in the true context. Some of the more prominent effects of segregation are considered below.

Due to the sensitivity of consolidation characteristics to changes in grainsize distribution, the stratified nature of the tailings results in serious limitations on the viability of accurate predictive modelling based on the results of laboratory testing alone.


Sand
Slimes
Beach
Clarification Pond
Beach
Discharge Points
Discharge Points
Sandy Tailings
Sandy Tailings
Slimes
Sand
Slimes
Sand
Slimes

GENERAL PATTERN OF SEGREGATION OF TAILINGS DEPOSITED HYDRAULICALLY IN A CONVENTIONAL IMPOUNDMENT AREA

FIGURE 2 - 1

Contrasts in permeability between adjacent layers result in not only strong anisotropic behaviour overall, but also possible local interlayer drainage. Combined with areal segregation, this creates a complex spatial drainage pattern in the field which is not readily simulated in predictive modelling.

. The concentration of fine-grained particles deposited by a process of sedimentation in quiescent water in the area of the clarification pond, results in deposited tailings characterized by very high initial water contents and void ratios and very low initial densities. Inherent with the very loose and fine-grained nature of the tailings in this area, it is also typified by high compressibility, low rates of consolidation and low strength. Consequently, such areas present the greatest difficulties in the design and implementation of reclamation measures.

In contrast, material deposited in the beach areas near the discharge points consists predominantly of the coarser sandy fraction of the tailings. Deposition on beach areas above the water level of the clarification pond also enhances drainage of water from the deposited tailings. Consequently, the tailings in such areas are considerably less compressible, more permeable and of higher strength compared to the tailings in the clarification pond area.

The contrasts in compressibility and rate of consolidation between the slime-predominant and sand-predominant areas arising from areal segregation of the tailings on deposition described above, accentuates the difficulties in maintaining the integrity of the surface cover and the effectiveness of the surface drainage measures in the long term due to large differential settlements.

2.2.2 Thickened Tailings Deposited Hydraulically

Deposition of thickened tailings slurry in thin layers by a distributed discharge system such as the subaerial method used at the Key Lake project (Knight and Haile, 1983), tends to reduce the degree of segregation compared to conventional hydraulic deposition. Some stratification and areal segregation probably still occurs as a result of variations in operational details. Initial water contents and void ratios on deposition tend to be high as in the case of conventional hydraulic deposition. However, more uniform conditions in this case would reduce the amount of differential settlements in the long term. Special measures in providing underdrainage and in using a cyclic discharge sequence to allow desiccation of the tailings to take place after deposition of each layer, are expected to promote the rate of consolidation of the tailings during the period of active deposition in this case.

2.2.3 Predewatered Tailings Deposited Mechanically.

The Pervious Surround Method of disposal currently being used at the Collins Bay B-zone project (Gulf Minerals, 1980, 1981) involves predewatering of the tailings and/or the neutralized raffinate precipitates codeposited with the tailings, by vacuum disc filters prior to deposition in the disposal area. This process is highly effective in removing the excess water from the tailings slurry and reduces the tailings to a semi-solid consistency which can be handled by haulage equipment or special pumping or conveying systems. For all practical purposes, no segregation during the process of deposition is anticipated in this case. The only variation in grainsize distribution within the deposited tailings would be due to minor changes in grind from time to time resulting from changes in ore conditions.

In view of the final submergence of the entire tailings deposit below the permanent natural groundwater table, the top cover is only required as a diffusion barrier and thus is practically insensitive to post-decommissioning consolidation settlements. On the other hand, the long term performance of the system relies on the absence of excess pore-water pressures within the tailings in the decommissioned state. The focus on a consolidation study in this case will therefore be on the pore-water pressure dissipation aspects rather than on settlements.

2.3 Loading Conditions and State of Stress

Consolidation, in the classical sense, normally refers to volume change of saturated soils as a result of applied stresses, and is governed by Darcy's law of fluid flow through porous media. One exception to this basic mechanism of saturated consolidation is the case where frost effects result in flow of the pore-water accompanied by consolidation of the unfrozen soil in the immediate vicinity of the freezing front. Thermal gradients and osmotic effects are believed to be the primary causes of consolidation in this case, as discussed later. Current developments in the consolidation theories and modelling techniques are mainly centred around the saturated consolidation as governed by fluid flow through porous media. This is also the principal area of interest in the present study. Some of the loading conditions and associated consolidation effects on the saturated tailings during various stages are examined in the following subsections.

2.3.1 Self-weight Stresses During Deposition

As mentioned earlier, accretion of the height of tailings during active deposition results in continuously increasing stresses and lengths of drainage paths concurrent with the progress of consolidation. The time dependent nature of the loading and drainage conditions results in a complex problem which cannot be analysed by the classical Terzaghi theory. The solution of this problem requires the use of computer based numerical methods described later in this report.

The rate of accretion relative to the rate of dissipation of excess porewater pressures during the deposition process, controls the state of consolidation at the end of the operation. It is therefore necessary to model this process of self-weight consolidation to enable post-deposition settlements to be estimated for consideration in planning decommissioning strategies. Such information is particularly important for scheduling the cover construction.

2.3.2 Post-deposition Recession of the Water Level

During hydraulic deposition, the large input of water not only maintains the water level at or above the tailings surface, but also causes the surrounding natural groundwater level to rise at the same time. After completion of deposition, the supply of water to the tailings area becomes drastically reduced. Surface drainage measures and top cover constructed during decommissioning further reduce the water infiltrating into the tailings deposit. Consequently, gradual lowering of water level in the long term is to be expected. The actual rate of decline and the ultimate steady-state position of the water level depends on a large number of factors including:

- meteorological conditions
- permeability of the tailings
- hydrogeologic conditions of the impoundment basin, and
- design of the impoundment dams and decommissioning works.

Accurate forecasting of the recession of the water level is virtually impossible due to complex interaction of all of the above factors. In general terms, however, the rate of decline of the water level in a uranium tailings deposit is anticipated to be very slow due to the low permeability of the tailings and the typically poor to very poor base and side drainage conditions created by initial site selection and design requirements for such facilities as discussed earlier. Similarly, the final position of the water level is anticipated to be generally close to the tailings surface.

The state of stress of the tailings deposit is closely related to the position of the water level. Lowering of the water level results in an increase of stresses in the tailings below it due to the loss of the buoyancy effect on the tailings within the zone of drawdown. At the same time, drainage of the tailings above the water level from an initially saturated state to an unsaturated condition results in negative pore-water pressures which increases the effective stresses and thus induces shrinkage and settlements. Lowering of the water level also exposes the upper part of the tailings to desiccation effects due to evaporation which may have far greater impact on the consolidation of the tailings than negative pore-water pressures due to gravity drainage, as discussed later. It will also be shown later that the lowering of the water level is the single most important natural process controlling the post-decommissioning settlement of a tailings deposit.

2.3.3 Superimposed Loads

Activities during decommissioning such as regrading of the tailings surface, surcharging operations and cover construction, produce long or short term changes in stresses within the underlying tailings and consequently, result in additional consolidation settlements. Because these superimposed loadings usually cover large areas and are relatively uniformly distributed, it is usually satisfactory to consider these as one-dimensional consolidation problems in modelling analyses. As indicated earlier in connection with cover construction, time scheduling of these activities relative to the end of tailings deposition and the progress of other processes such as lowering of the water level and desiccation, has important bearing on the resulting consolidation settlements and must be taken into consideration in predictive modelling studies.

2.4 Thermal Effects

In the Canadian environment, freezing and thawing of tailings during and following deposition interfere with the normal process of consolidation. Thermodynamic processes also play a prominent role in the moisture movement through the tailings, which in turn leads to stress and volume changes. Such processes are therefore of primary interest in the study of post-decommissioning consolidation of uranium tailings, and are examined in general terms below.

2.4.1 Desiccation

The process of natural desiccation of soils relies on the removal of water by evaporation, thereby generating capillary potential gradients which causes upward moisture migration to sustain the process. Gardner, et al (1922) defined capillary potential at a point as the work required to move a unit mass of water from a point of zero potential to the point considered. Gardner (1920) showed from some experimental data that the capillary potential varied linearly with the reciprocal of the moisture content for a wide range of moisture contents and expressed the empirical relationship in the form:

$\psi = (a/w) + b$

in which,

ψ = capillary potential

w = moisture content

a, b = empirical contents

During the initial stage of desaturation when the moisture content is high, the capillary potential is mainly a function of surface tension of water and the curvature of the air-water interface, and thus depends on the pore size distribution of the soil (Gardner, 1960). However, on further desaturation with reduction in the moisture content accompanied by increase in capillary potential, mechanisms other than the above capillary model become predominant. Gardner (1960) indicated that for swelling clays, the capillary model is inadequate even at very low suctions and for most soils, it probably fails at suctions above one bar (approximately 1 atmosphere or 100 kPa). The thermodynamic nature of the mechanism controlling moisture movements, effective stresses and volume change in soils at relatively high suction or low degrees of saturation, was recognized by Edlefsen and Anderson (1943), who defined the potential of pore-water in terms of its free energy, "f" in the expression:

$f = h - Ts$

in which,

h = the heat content

T = absolute temperature

s = the entropy

Croney, et al (1958) also employed thermodynamic principles to study soil moisture movement and distribution and illustrated the effect of temperature and temperature gradient on moisture migration.

Because of the effectiveness of evaporation in reducing the moisture content of a soil near its surface to very low degrees of saturation, very high negative pore-water pressures can be generated which in turn result in very high effective stresses and strength, accompanied by large reductions in volume. For instance, Gardner (1960) indicated that the wilting point for most soils occurs at a suction of about 15 bars which is an order of magnitude higher than the maximum 1 bar of suction which can be generated by gravitational capillary effects due to lowering of the water level. Common observations of the hardening and shrinkage cracking of clays on drying are good evidence of this fact. Recognition of evaporative desiccation as the principal mechanism controlling strength and volume change of soils near the surface, is of great importance in the study of post-deposition consolidation of tailings deposits, particularly those deposited hydraulically. This is because of the very high void ratios of the tailings near the surface as a result of subaqueous sedimentation and self-weight consolidation during deposition, which could be as high as 5 or more for fine-grained uranium tailings slimes. Shrinkage due to evaporative desiccation could reduce the void ratios to a range of 1 to 2 and thus desiccation of even a thin upper layer of the tailings to depths of 1 or 2 m below the surface can have a large effect on surface settlement in the long term, as demonstrated by the results of this study described later in this report.

Although a considerable amount of research work has been carried out on moisture movement in unsaturated soils, mainly in the field of agronomy and the study of movements of pavements as a result of moisture migration in the subgrade soils, little is known about the rate and maximum depth of desiccation in tailings in general and uranium tailings in particular. Theoretical relationships linking various gravitational, surface tension and thermodynamic parameters to capillary potential and thus to moisture movement, strength and volume change phenomena are useful in indicating the general nature and trends of influence of these parameters in the overall process. However, the sensitivity of the capillary potential to changes in pore-size distribution coupled with the stratified nature of deposited tailings, and the difficulty in defining the time dependent variations of the thermodynamic parameters, all contribute to the need for an empirical approach in practice in the case of predictive modelling of evaporative effects on uranium tailings, which is an important part of the study reported herein.

2.4.2 Freezing of Unsaturated Tailings

Formation of ice lenses in unsaturated fine-grained soils due to freezing results in temperature and vapour pressure gradients accompanied by heat transfer and moisture migration (Croney and Coleman, 1960). Frost damage to road pavements has been attributed to such effects (Croney, 1952).

Moisture movement due to ice formation results in consolidation of the adjacent unfrozen soil. At the same time, any consolidation process originally on-going within the frozen zone, is arrested. Subsequent thawing results in not only resumption of the original consolidation process but also releases the excess water built up locally in the form of ice. The complex interaction of a thermal process of heat transfer from surface and/or groundwater sources with consolidation makes it virtually impossible to predict the rate of release of free water during thaw.

2.4.3 Freezing of Saturated Tailings

Hydraulic deposition of tailings in cold weather results in freezing of saturated tailings in beach areas above the clarification pond. A part of the free water within the tailings slurry, which normally drains from the deposited tailings in beach areas into the clarification pond in non-freezing weather, may also become frozen, forming layers or lenses of ice. In addition, induced movement of pore-water towards the freezing front from the unfrozen tailings below could also occur, resulting in increasing thickness of segregated ice accompanied by consolidation of the underlying tailings. Frost heaving due to such pore-water movement and formation of segregated ice is a well known phenomenon in soil mechanics although the actual mechanism of the process is not fully established. In the case of tailings in which the pore-water is highly saline, however, it is believed that osmotic effects could at least play a significant part in the process.

Depending on the rate of accretion, part of the frozen tailings with the entrained ice may remain frozen in the long term due to an insulating effect of the tailings deposited subsequently. As in the case of unsaturated tailings discussed above, prediction of the extent and rate of thawing of the frozen tailings is virtually impossible. Because of the comparatively larger volume of saturated tailings and entrained ice which could accumulate over the life of a tailings deposit, the impact on the consolidation process in this case will be considerably greater than freezing of the unsaturated tailings near the surface.

Although the impact due to freezing on consolidation of tailings is difficult to quantify, the nature of its influence needs to be taken into consideration in any consolidation study. If the frozen tailings remains as permafrost, the overall compressibility of the tailings in the beach area will be even lower than if freezing did not occur. The long term differential settlements between the clarification pond area (where no frozen tailings is expected to occur) and the beach areas will be accentuated in that event. However, if gradual thawing did occur in the long term, ultimate settlements in the beach area could exceed those in the clarification area, depending on the amount of entrained ice initially trapped in the tailings.

2.5 Chemical Processes

The milling and ore extraction processes not only determine the grind (grainsize distribution) of the tailings, but also produce a variety of chemical compounds which are discharged with the tailings into the disposal area. Some of these chemicals remain in solution in the pore-water and the balance exist in solid form intimately mixed with the tailings. In either form these chemicals could have significant influence on the consolidation behaviour of the tailings after deposition.

As already mentioned, the chemical laden pore-water could partly account for consolidation of tailings adjacent to a freezing front in cold weather. The salinity of the pore-water also influences its viscosity, vapour pressure, freezing point and surface tension, which all directly or indirectly influence the consolidation processes in saturated and unsaturated states.

Chemical precipitates coexisting with the tailings could impart unusual characteristics to the mixture, which are not representative of the tailings alone. In acid leach processes, for example, neutralization of the tailings by lime treatment prior to disposal results in the formation of gypsum which precipitates out of solution in needle like crystals (laths). Although these crystals are usually coarser in particle size than clay minerals, their elongated shape could result in a very loose meta-stable structure if they are present in sufficient quantities in the tailings and thus give rise to a clay-like behaviour. Precipitates of heavy metal hydroxides in the tailings from some high-grade ores are also very fine-grained and tend to impart clay-like properties to the tailings when present in significant quantities.

3.0 THEORIES OF CONSOLIDATION

The term "consolidation" was first used by Terzaghi to describe a time dependent phenomenon involved in the deformation of saturated fine-grained soils under an externally applied load. Although a number of simplifying assumptions were made in his development of the theory, Terzaghi's initial treatment of the problem has been found to be adequate for most foundation engineering applications and is universally used for predicting foundation settlements. However, indiscriminant use of the method without considering the limitations arising from the basic assumptions vital to the formulation of the theory, could lead to serious errors in predictions or inadequate appreciation of the degree of uncertainty involved in such predictions. It is therefore appropriate to review at this point, the basic assumptions involved in the Terzaghi theory. These assumptions are as follows:

- the consolidating layer has a constant quantity of solid particles (i.e. no accretion or depletion of the consolidating material is involved during the process of consolidation)
- the consolidating material is fully saturated
- material solids and pore water are perfectly incompressible
- flow of pore-water strictly obeys Darcy's law
- the consolidating material has been fully consolidated under its own weight
- external loading is the only cause of consolidation
- strains resulting from external loading are infinitesimal, consequently the permeability and the lengths of internal drainage paths remain constant during the progress of consolidation
- the consolidating material is laterally confined and uniformly stressed in any horizontal plane so that consolidation takes place in one dimension only.

In reality, of course, most of these assumptions are not strictly valid and proper application of the theory and interpretation of the results of predictive modelling requires a sound appraisal of the degree of approximation to which the actual conditions are simulated.

Over the years since the postulation of Terzaghi's original theory, many new theories have been advanced to enable analysis of special situations in which one or more of the above assumptions are invalid, to be carried out with better accuracy. A complete review of all these theories is not necessary for the purpose of this study. It is more productive and appropriate to focus our attention on those developments which are directly pertinent to the problem of the consolidation of uranium tailings in the Canadian setting.

Given the nature of the problem described earlier in Section 2.0 of this report, it is evident that theories suitable for use in predictive modelling of the consolidation behaviour of a uranium tailings deposit will have to take proper account of the following phenomena:

- self-weight consolidation due to accretion of tailings during active deposition,
- large-strain and non-linear behaviour of the tailings during consolidation, and
- desiccation or unsaturated consolidation, at least in the post-decommissioning long term condition.

There are three basic conditions which could lead to departure from the one-dimensional consolidation condition assumed in the classical and most of the modern day theories. These are:

- non-uniform or concentrated external loading conditions
- boundary drainage conditions, and
- spatial variations of properties of the consolidating material.

In the case of conventional hydraulically deposited tailings, it has been shown that loading and boundary drainage generally satisfy the conditions required for one-dimensional consolidation in the vertical direction. Spatial variations of properties do occur, however, due to segregation during deposition. In special cases such as the Pervious Surround Method (Matich and Tao, 1984) used at the Collins Bay B-zone project and the Sub-aerial Method used at the Key Lake project, conditions provided by design are favourable to three-dimensional drainage of the deposited tailings. Current developments in theories dealing with such three-dimensional effects (Biot, 1972 and Carter, Small and Booker, 1977) are not sufficiently advanced for application in practice (Schiffman, Pane and Sunara, 1985). This study will therefore be limited to the application of one dimensional consolidation theories, although the possible influence of three-dimensional effects are to be borne in mind.

The consolidation theories directly pertinent to the present study are discussed in the following subsections.

3.1 Consolidation of Saturated Materials

There are three classes of theories dealing with one-dimensional consolidation of saturated materials, which include the effect of time dependent loading conditions.

3.1.1 Infinitesimal Linear Strain Theory of Accreting Layer

A first attempt in the formulation of a consolidation theory to account for self-weight consolidation of an accreting layer was published by Gibson (1958) who modified the original differential equation given by Terzaghi in the form:

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2}$$

by adding a term $\gamma'\frac{dh}{dt}$ representing the effect of the self-weight of the accreting material which is undergoing consolidation, to the form:

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} - \gamma' \frac{dh}{dt}$$

In these equations,

u = excess pore-water pressure

t = time

z = Eulerian coordinate of a point above the base of the consolidating layer

γ' = buoyant density of the consolidating material

h = thickness of the consolidating layer at time "t"

c_v = coefficient of consolidation

Solutions of the above general differential equation were obtained for the following special cases:

- continuous deposition on an impermeable base at a rate of accretion proportional to the square root of elapsed time from the beginning of the process of deposition,
- continuous deposition on an impermeable base at a constant rate of accretion proportional to the elapsed time from the beginning of the process of deposition
- two other similar cases as above on a permeable base.

A numerical method using a finite difference form of the general differential equation was also described by the author for solving problems with variable rates of accretion.

Because of the infinitesimal strains and linear properties assumed in the formulation of this theory, it is subjected to the same limitations in application as the classical Terzaghi theory except its capabilities in dealing with self-weight consolidation. Its application in the present study is therefore limited to serve an illustrative purpose for comparison with the more rigorous finite strain theories. In the special case of deposition of predewatered tailings with relatively low initial void ratios, such as in the Collins Bay B-zone project, however, application of this theory may be quite acceptable due to the comparatively small strains involved, and merits consideration due to the simplicity of the numerical procedures compared to the more rigorous theories.

3.1.2 Linearized Finite Strain Theories

Because of the large deformations that take place during consolidation of very loose deposits, it is necessary to refer to the soil skeleton as a time independent frame of reference in mathematical modelling of the consolidation process. This need gives rise to the "reduced coordinate" (sometimes referred to as the "material coordinate") usually used in the formulation of finite strain consolidation theories. These are imaginary dimensions computed as the distance between two points in the soil skeleton by reducing its void ratio to zero. Two other coordinate systems frequently referred to in finite strain mathematics, are the "Lagrangian coordinate" which is the Eulerian distance between two points in the soil skeleton at the initial void ratio prior to the start of the consolidation process, and the "convective coordinate" which is a time dependent Eulerian distance between two points in the soil skeleton during the progress of consolidation. Because the base of a consolidating layer can usually be assumed to be spatially fixed, it is usually used as the datum plane for the above coordinate systems. In that case, the Lagrangian height of a point is the initial height of the point before consolidation begins and the convective height of the same point traces its settlement in time as computed by the theory of consolidation. The reduced height of the same point is the height of material solids between the point and the datum plane, which remains constant throughout the consolidation process.

Rigorous formulation of the non-linear finite strain theory was given by Gibson, et al (1967). However, because of its highly non-linear nature no practical solution of the general differential equation derived by the authors was advanced for some time.

Gibson, et al (1981) introduced approximations to linearize the general differential equation to the form

$$\frac{\partial^2 e}{\partial z^2} - \lambda(\gamma_s - \gamma_w)\frac{\partial e}{\partial z} = \frac{1}{g}\frac{\partial e}{\partial t}$$

where,

$$g = -\frac{k}{\gamma_w}\,\frac{1}{1+e}\,\frac{d\sigma'}{de}$$

was assumed to be constant and is generally referred to as the finite strain coefficient of consolidation analogous to c_v in the conventional infinitesimal strain consolidation theory. The authors suggested that a reasonable approximation of typical experimental consolidation characteristics could be expressed in the form

$$g = c_v/(1 + e_a)^2$$

where e_a is the void ratio at the average stress $'_a$ during the consolidation process.

To further linearize the differential equation, the function

$$\lambda = -\frac{d}{de}\,\frac{de}{d\sigma'},$$

is assumed to be constant and is approximated by

$$\lambda = \frac{1}{\sigma'_a} \log_e \left[\frac{e_a - e_m}{e_{oo} - e_m}\right]$$

where,

e_{oo} = void ratio at zero effective stress

e_m = void ratio at maximum effective stress during the consolidating process

e_a = void ratio at average effective consolidating stress

A computer based finite difference method (FSCON2-I) for solving the above linearized differential equation for single and double drainage boundary conditions is given by Cargill and Schiffman (1979).

A non-dimensional form of the general linearized differential equation,

$$\frac{\partial^2 E}{\partial Z^2} + N \frac{\partial E}{\partial Z} = \frac{\partial E}{\partial T}$$

where,

$Z = z/\ell$

ℓ = material thickness (reduced coordinate) of the consolidating layer

$E(Z,T) = e(z,t)/e(0,0)$

$T = gt/\ell^2$

$N = \lambda \ell (\gamma_s - \gamma_w)$

is solved by a computer program FSCON1-I (Cargill and Schiffman, 1980). Dimensionless charts prepared from solutions from this program relating degree of consolidation settlement to T and N for various boundary conditions are given by Cargill (1984).

It should be noted that all of the above methods are applicable to the consolidation of a non-accreting layer which is initially fully consolidated under self-weight and any surcharge loading which may be acting on its upper surface prior to the application of an increment of surcharge loading which initiates the consolidation process being analysed by these methods. An upgraded version (SCHF81) of FSCON1-I and FSCON2-I has been developed by Nikow, Sunara and Schiffman (1985) which takes into account both self-weight consolidation of a layer accreting at a variable rate and consolidation due to externally applied surcharge loading.

There are numerous other efforts in published literature in solving finite strain consolidation problems using various linearizing techniques, including Been and Sills (1981), Monte and Krizek (1976), Somogyi (1980) and Lehner (1984). It is not intended to carry out a complete review of all of these methods in this study. Because of the complex interaction of different processes as discussed later, direct application of any one of these methods by itself will not be adequate for prediction of long term consolidation behaviour of an uranium tailings deposit. The brief review given above is intended to introduce the basic principles involved in the study of finite strain consolidation problems.

3.1.3 Non-linear Finite Strain Theory

Cargill (1985) expressed the general differential equation governing non-linear finite strain consolidation originally published by Gibson, England, and Hussey (1967) in the form:

$$\left(\frac{\gamma_s}{\gamma_w} - 1\right) \frac{d}{de} \frac{k(e)}{1+e} \frac{\partial e}{\partial z} + \frac{\partial}{\partial z} \frac{k(e)}{\gamma_w(1+e)} \frac{d\sigma'}{de} \frac{de}{dz} + \frac{\partial e}{\partial t} = 0$$

where,

γ_s = unit weight of solids
γ_w = unit weight of water
e = void ratio
$k(e)$ = coefficient of permeability as a function of void ratio
z = vertical material coordinate measured against gravity
σ' = effective stress
t = time

No closed form analytical solution of the above differential equation is available. Instead, an explicit finite difference scheme was used by the author to solve the problem. In this method the governing equation was converted to a finite difference form, as follows:

$$e_{i,j+1} = e_{i,j} - \frac{\tau}{\gamma_w}\left\{\left[\gamma_c \beta(e_{i,j}) + \frac{\alpha(e_{i+1,j}) - \alpha(e_{i-1,j})}{2\delta}\right]\frac{e_{i+1,j} - e_{i-1,j}}{2\delta} + (e_{i,j})\,\frac{e_{i+1,j} - 2e_{i,j} + e_{i-1,j}}{\delta^2}\right\}$$

where,

τ = time interval in finite difference mesh
γ_c = buoyant unit weight of solids
$\beta(e)$ = a function of void ratio and permeability
= $\frac{d}{de}\left[k(e)/(1+e)\right]$
$\alpha(e)$ = a function of void ratio, permeability and compressibility
= $k(e)/(1+e)\,\frac{d\sigma'}{de}$
= vertical space interval in material coordinates in the finite difference mesh

A computer program PCDEDF is used to execute this finite difference solution of the consolidation process, in which $\alpha(e)$ and $\beta(e)$ are obtained directly from laboratory curves included as data input. Thus, no curve fitting or other approximations to linearize the general differential equation are involved. The method therefore provides a more accurate solution of the finite strain consolidation process compared to the linearized approach.

3.2 Consolidation of Unsaturated Soils

Consolidation of unsaturated soils could take place as the result of

- externally applied loading, or
- thermodynamic processes

Because of the complexity of the processes involved, theories or methodologies for predicting such consolidation behaviour are necessarily empirical to semi-empirical in nature. It is also important to note that more than one process could occur simultaneously within the soil and therefore recognition of the predominant process is essential to enable reasonably accurate forecasting of the consolidation behaviour by selecting the correct predictive methodology.

3.2.1 Externally Applied Loading

Fredlund and Morgenstern (1976) described an element of unsaturated soil as a four-phase system with two phases (soil particles and air-water interface or "contractile skin") which undergo deformation under applied stresses and two phases (pore-air and pore-water) which flow under applied pressure gradients. Stress state parameters in the form $(\sigma - u_a)$ and $(u_a - u_u)$

where σ is the total stress, u_a the pore-air pressure and u_w the pore-water pressure, were suggested as the independent variables governing monotonic deformation of the soil by Fredlund and Morgenstern (1976). Constitutive relations linking these stress state variables and deformation of an element of unsaturated soil were proposed as follows:

(a) volumetric strain of the soil structure,

$$\epsilon = \frac{1}{v}\frac{\partial v}{\partial(\sigma - u_a)} d(\sigma - u_a) + \frac{1}{v}\frac{\partial v}{\partial(u_a - u_w)} d(u_a - u_w)$$

(b) net volume change of water phase due to inflow or outflow from the element of soil,

$$\theta_w = \frac{1}{v}\frac{\partial v_w}{\partial(\sigma - u_w)} d(\sigma - u_w) + \frac{1}{v}\frac{\partial v_w}{\partial(u_a - u_w)} d(u_a - u_w)$$

where,

v_w = volume of water in the element

The semi-empirical nature of these constitutive relations was emphasized by the authors. It would thus be necessary to verify the uniqueness of these relations on a case by case basis, particularly because of the unpredictable behaviour of different soil types. For example, the authors cited evidence from Barden et al (1969) that a decrease in suction (u_a-u_w) in a stable structured soil will result in swelling whereas it may cause a volume decrease in a meta-stable soil. Because of hysteresis effects, the constitutive relations are non-unique where stress reversal occurs due to loading and unloading or wetting and drying. For monotonic deformations however, evidence of uniqueness of the constitutive relations was presented by the authors for Regina clay and kaolin under high degrees of saturation and small stress and volume changes. Similar evidence of uniqueness under monotonic stress paths and non-unique behaviour upon stress reversal was also presented by Matyas and Radhakrishna (1968).

Fredlund and Hasan (1979) presented a one-dimensional consolidation theory for unsaturated soils based on the above constitutive relations. The theory requires the conventional assumptions as Terzaghi's theory with the following additional assumptions:

i) the coefficients of permeability with respect to water and air and the volume change moduli remain constant during the transient processes, and

ii) the effects of air diffusing through water and the movement of water vapour are negligible.

To determine ultimate volume changes, experimentally determined constitutive surfaces defining the soil structure and water phase compressibility as functions of $(\sigma - u_a)$ and $(u_a - u_w)$ are required. From these constitutive relations, the volumetric strains are given by

$$\Delta V/V = m_1^s\, d(\sigma - u_a) + m_2^s\, d(u_a - u_w) \text{ for soil structure}$$

$$\Delta V_w/V = m_1^w\, d(\sigma - u_a) + m_2^w\, d(u_a - u_w) \text{ for water phase}$$

where,

m_1, m_2 = compressibility with respect to $(\sigma - u_a)$ and $(u_a - u_w)$ respectively,

and the superscripts indicate soil structure and water phase, respectively.

Differential equations governing the progress of consolidation in terms of dissipation of pore-water and pore-air pressures were given respectively as

$$\frac{\partial u_w}{\partial t} = - C_w \frac{\partial u_a}{\partial t} + c_v{}^w \frac{\partial^2 u_w}{\partial y^2}$$

$$\frac{\partial u_a}{\partial t} = - C_a \frac{\partial u_w}{\partial t} + c_v{}^a \frac{\partial^2 u_a}{\partial y^2}$$

where the subscript and superscript, "a" and "w", indicate air phase and water phase, respectively, and

C_w = interactive constant for water phase

= $(1-m_2{}^w/m_1{}^w)/(m_2{}^w/m_1{}^w)$

C_a = interactive constant for air phase

$$= \left[\frac{m_2{}^a}{m_1{}^a}\right] \Bigg/ \left[(1 - \frac{m_2{}^a}{m_1{}^a}) + \frac{(1 - S)n}{(u_a+u_{atm})m_1{}^a}\right]$$

$c_v{}^w$ = conventional coefficient of consolidation for water phase
= $(1/R_w)(k_w/\gamma_w\ m_1{}^w)$

$c_v{}^a$ = coefficent of consolidation for air phase

$$= \frac{DR\theta}{W} \frac{1}{(1-R_a)(u_a + u_{atm})\ m_1{}^a + (1-S)n}$$

D = coefficient of air diffusion subjected to an air pressure gradient

R = universal gas constant

θ = absolute temperature

w = molecular weight of the mass of air

S = degree of saturation

n = porosity of the soil

An explicit finite difference scheme was given by the authors for solving the differential equations in the form

water phase:

$$u_w(i,j+1) = u_w(i,j) - C_w u_a(i,j+1) - u_a(i,j) + c_v^w \frac{\Delta t}{(\Delta y)^2} \; u_w(i+1,j) + u_w(i-1,j) - 2\, u_w(i,j)$$

air phase:

$$u_a(i,j+1) = u_a(i,j) - C_a u_w(i,j+1) - u_w(i,j) + c_v^a \frac{\Delta t}{(\Delta y)^2} \; u_a(i+1,j) + u_a(i-1,j) - 2\, u_a(i,j)$$

3.2.2 Thermodynamic Processes

3.2.2.1. Desiccation

The most prominant thermodynamic process related to consolidation of soils is desiccation due to evaporation. Removal of pore-water by evaporation can reduce the degree of saturation in soils to very low values, far below those possible by gravity drainage. Although it is well known that pore-water can sustain extremely high tensile stresses, it does not mean that such tensile stresses can be transmitted hydraulically by lowering of the water level by gravity drainage. There is evidence that negative pore-water suction induced by gravity drainage is limited to a maximum of about 1 bar (approximately 100 kPa) whereas suctions in excess of 100 bars could be induced by desiccation effects. Thus consolidation effects on the upper part of a soil deposit subjected to evaporation are for all practical purposes, determined entirely by moisture movements governed by thermal and vapour pressure gradients. Although the nature of the thermodynamic principals governing such moisture movements is known in general terms, determination of parametric values and application of the theories in practice are difficult and not warranted for the purpose of this study. Instead, it is considered appropriate to use an empirical approach proposed by Cargill (1985) as described below.

The case analysed by the author was that of the desiccation of a hydraulically deposited dredged material after complete decantation of supernatent water. This case is parallel to that of the desiccation of a hydraulically deposited uranium tailings deposit and therefore is ideally suitable for application in the present study.

A simplistic but rational water balance approach was used by the author to compute the net volume change and thus the settlement due to desiccation of the dredged material. An important feature of this method is the coupling of the desiccation process of the upper part of the material to the ongoing consolidation process of the underlying saturated fill.

Based on experimental work carried out by Brown and Thompson (1977), Haliburton (1978) and Gardner and Hillel (1962), a two stage process was used to describe the evaporative drying of a dredged material. The net loss of water from the surface of the soil during Stage 1 drying was expressed as

$$W_1 = C_{E1}E_p - (1 - C_D)R - W_C$$

where,

C_{E1} = evaporation efficiency during Stage 1 drying
E_p = Class A pan evaporation loss
C_D = surface drainage efficiency
R = rainfall
W_C = water supplied to the surface by the consolidating material below the crust.

At the beginning of the drying process, the surface of the material was considered to be fully saturated at an initial void ratio of "e_{oo}" corresponding to an unconsolidated condition under zero effective stress (i.e. the end point of the sedimentation process and the starting point of a soil formation). Loss of water by evaporation initiates the first stage drying where the soil remains saturated while undergoing a shrinkage in volume equal to the volume of water loss. The author suggested a limiting void ratio "e_{sL}" as the end point of Stage 1 drying and a maximum depth of the zone of Stage 1 drying given by

$$h_1 = (L - Z_{SL})(1+e_{SL})$$

where

L = material coordinate of the surface of the layer

Z_{SL} = material coordinate of the point of intersection of e_{SL} and the ultimate void ratio distribution curve under self-weight consolidation as illustrated in Fig. 3-1 below.

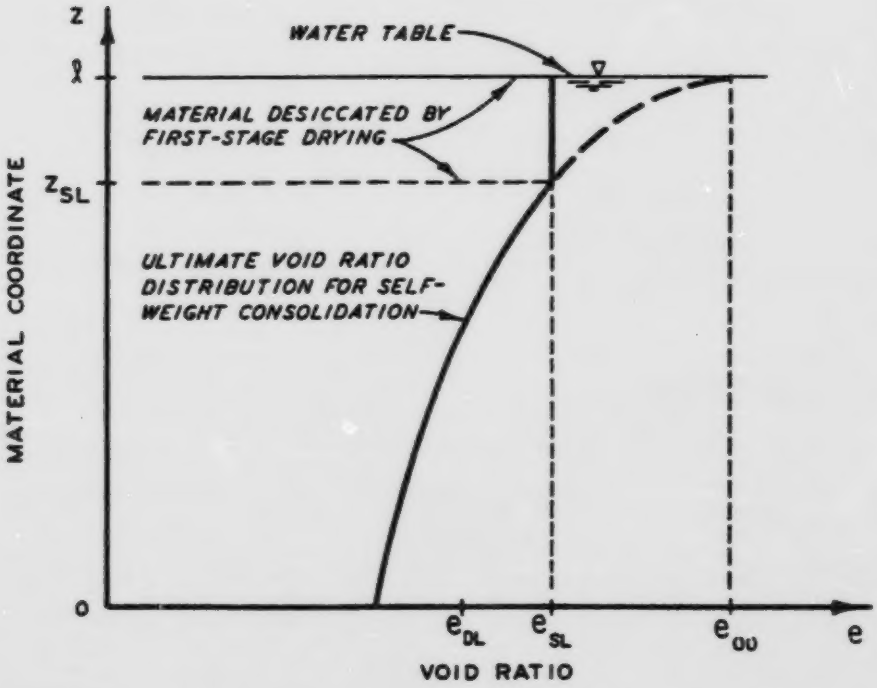
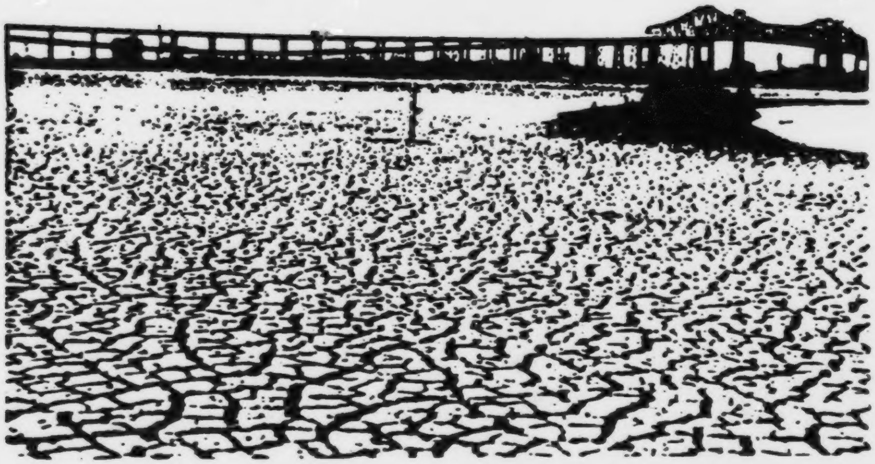

z
ℓ
WATER TABLE
MATERIAL DESICCATED BY FIRST-STAGE DRYING
z_{SL}
ULTIMATE VOID RATIO DISTRIBUTION FOR SELF-WEIGHT CONSOLIDATION
MATERIAL COORDINATE
0
e_{DL}
e_{SL}
e_{00}
e
VOID RATIO

Figure 3-1. Maximum depth of material desiccated by first-stage drying (Cargill, 1985)

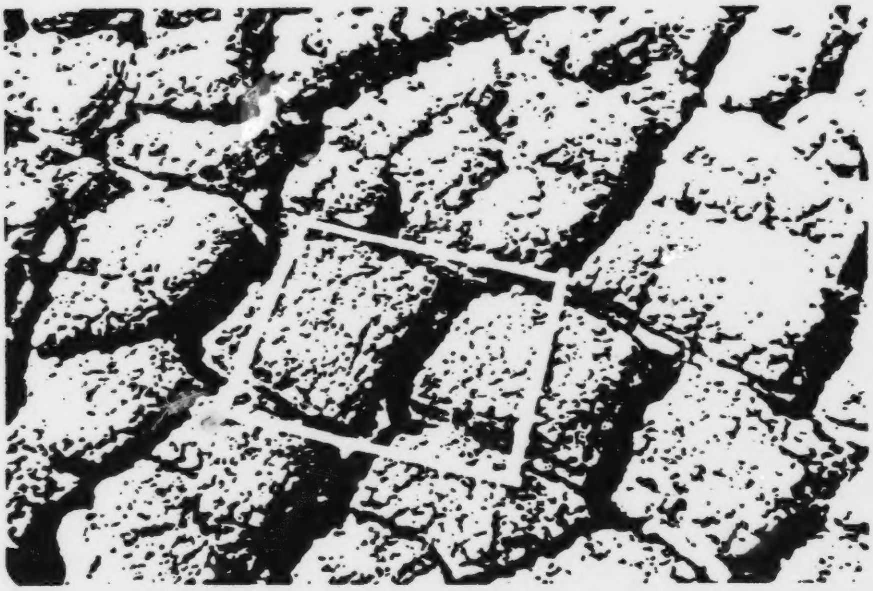
During the Stage 1 drying the evaporation efficiency remains essentially constant at or slightly below 1.0 (i.e. a constant fraction of the Class A pan evaportion loss).

The value of e_{sL} can be measured in the laboratory by determining the void ratio at which point an initially saturated sample starts to desaturate when allowed to air dry under zero or very low confining and vertical stresses which reasonably simulate the upper part of the soil subjected to drying under field conditions. Based on the work by Haliburton, it was suggested that e_{sL} could be taken as that corresponding to a water content 1.8 LL where LL is the liquid limit of the material.

After Stage 1 drying has reached the maximum depth defined above, desaturation of the soil starts at the surface of the soil accompanied by formation of surface cracks (Fig. 3-2) and reduction in volume towards a limiting value designated by the author as e_{DL}, the desiccation limit. The volume of voids at this stage include water filled pore space within intact blocks of solid and air filled voids which include both the air voids within the intact blocks and the volume of cracks between them. The nature of the parameter e_{DL} is therefore somewhat different from the void ratio at the shrinkage limit normally considered in soil mechanics. The formation of shrinkage cracks is the result of three-dimensional consolidation due to desiccation and is a field scale effect which cannot be determined by laboratory methods. Accurate measurement of the volume of cracks in the field of course is not possible. However, by estimating the average volume of cracks and including it in the value of e_{DL}, the author effectively reduced a three-dimensional consolidation phenomenon into an equivalent one-dimensional consolidation for settlement calculation purposes. In the absence of factual information on e_{DL}, the author suggested a value corresponding to a water content equal to 1.2 PL where PL is the plastic limit of the soil, could be used as a guideline in selecting an assumed parametric value for preliminary estimating purposes.



a. View of settlement plate No. 4



b. Reference scale is approximately 18 in.^{2}

Figure 16. Drum Island disposal area

Figure 3-2 Surface cracks formed in dredged material due to desiccation. (Cargill, 1985)

During second stage drying the evaporation efficiency, C_{E2}, decreases with time and depth of the water table below the surface of the soil. For simplicity, considering the approximate nature of the analysis, the author suggested that this could be taken as a linear function of the water table depth expressed in the form

$$C_{E2} = C_{E1}\left(1 - \frac{h_w}{h_2}\right)$$

in which C_{E1} is the evaporation efficiency during first stage drying, h_w is the average depth to the water table for the time interval under consideration and h_2 is the maximum depth of the second stage drying. The net water lost during the second stage drying can then be expressed in the form

$$W_2 = C_{E1}\left(1 - \frac{h_w}{h_2}\right) E_P - (1 - C_D)R - W_C.$$

The maximum depth of the second stage drying is determined by either the surrounding natural groundwater system which may limit the depth of lowering of the water level in the consolidating soil deposit under consideration to a certain value, or the point of intersection between e_{DL} and the ultimate void ratio profile due to self-weight plus surcharge due to lowering of the water level, as shown in Fig. 3-3, whichever is the less.

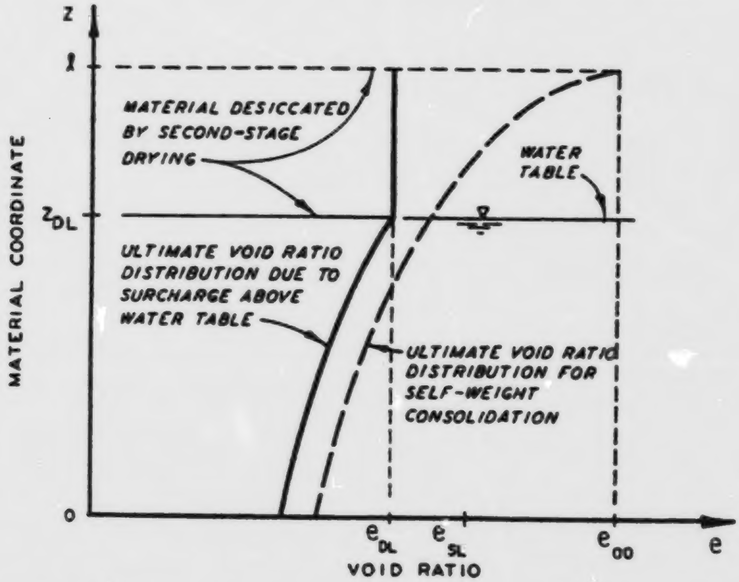

z
ℓ
MATERIAL DESICCATED BY SECOND-STAGE DRYING
WATER TABLE
z_{DL}
ULTIMATE VOID RATIO DISTRIBUTION DUE TO SURCHARGE ABOVE WATER TABLE
ULTIMATE VOID RATIO DISTRIBUTION FOR SELF-WEIGHT CONSOLIDATION
MATERIAL COORDINATE
0
e_{DL}
e_{SL}
e_{00}
e
VOID RATIO

Figure 3-3 Maximum depth of material desiccated by second-stage drying. (Cargill, 1985)

From the water balance calculations, the desiccation settlements were given as

$$\delta_{D1} = W_1 \qquad \text{for Stage 1 drying}$$

$$\delta_{D2} = W_2 - (1 - \frac{S}{100})\, h_w \qquad \text{for Stage 2 drying}$$

where

S = gross degree of saturation of soil above the water level

3.2.2.2 Frost Effects

As in the case of desiccation, only the general nature of the thermodynamic processes in moisture movements due to freezing in soils is known (Jumikis, 1962, 1957). Predictive methodologies available are mainly involved in depths of frost penetration, such as those quoted by Jumikis (1962). No known practical methods are available for quantitative prediction of the formation of segregated ice and associated consolidation of adjacent soils.

4.0 MODELLING METHODOLOGY

4.1 Basic Requirements

Considering the overall nature of the consolidation problem involving a uranium tailings deposit, the mathematical modelling methodology selected in the study of such a problem should ideally meet all of the following requirements:

- method based on finite strain theory
- full consideration given to variable permeability and non-linear compressibility characteristics of the consolidating material
- capable of dealing with self-weight consolidation of an accreting layer
- capable of dealing with effects of desiccation as a coupled phenomenon with on-going consolidation of saturated material below the desiccating crust
- capable of analysing the effects of surcharge loading placed over the surface of the consolidation material before or after the onset of desiccation
- capable of simulating base drainage conditions ranging from impermeable to semi-permeable to fully drained.

Numerical procedures involved in solving the finite strain consolidation problem constrain the selection of modelling methodology to available existing computer programs, including the following:

- FSCON1-I (Cargill and Schiffman, 1980)
- FSCON2-I (Cargill and Schiffman, 1979)
- SCHF81 (Nikou, Sunara and Schiffman, 1985)
- FSCON4 (Pane and Schiffman, 1980)
- TRUNC (Fayer and McKeon, 1984)
- PCDDF (Cargill, 1985)

Of these, only TRUNC and PCDDF can handle coupled saturated-unsaturated consolidation which is an indispensable requirement in the present study considering the intricate interaction between the two processes. TRUNC, however, is based on the infinitesimal strain theory for analysing the part of the problem involving saturated consolidation and deals only with unsaturated consolidation due to external loading and negative pore pressures generated by gravity drainage without considering the much more prominent effect due to evaporative desiccation. This then leaves PCDDF as the only choice for use in the present study since it considers the important effects of evaporative desiccation. A more detailed examination of this program is presented below.

4.2 The Program PCDDF (Cargill 1985)

4.2.1 Positive Features

This program is well documented in Cargill (1985), containing not only a detailed account of the basic theories, program listing and a complete manual of input and output facilities but also field verification case records on three full scale operating dredged material disposal sites. Analysis of saturated consolidation is based on non-linear finite strain theory and that of desiccation consolidation by empirical treatment of evaporation effects. Integration of the interaction of consolidation and desiccation effects into the computation subroutines allows full account of the water budget within the zone of desiccation to be taken while enabling a rational estimate of the recession of the water level below the surface of the consolidating material to be made. This in turn, allows the time dependent loading due to the surcharging effect arising from lowering of the water table to be defined and thus enables its effect on consolidation of the underlying tailing to be computed. Base drainage condition simulated by input provision for a foundation material with any specified permeability and drainage path length, provides the desirable flexibility of modelling actual field conditions. This program therefore meets nearly all of the basic requirements stipulated earlier.

4.2.2 Limitations

One severe limitation experienced in application of this program in the present study is the lack of input provision for modelling surcharge effects by external loading placed on the surface of the consolidating material. Material accretion is restricted to step-wise incremental placement of one type of consolidating material at its initial unconsolidated state (e_{oo}) only. Simulation of surcharging or cover placement is therefore difficult particularly in the case of deferred construction after allowing surface desiccation of the tailings to take place.

Hydrostatic pressures in the foundation soil are assumed to respond fully to the water level in the consolidating material. This prevents the "dry" base or fully drained base condition to be simulated.

The empirical nature of the input parameters controlling the desiccation process require field verificaiton to validate the parametric values. This is particularly significant considering the sensitivity of predicted settlements to these input parameters, as illustrated later in this study report.

4.2.3 Validation for Present Application and Parametric Study

To verify the applicability of the program PCDDF in analysing the consolidation behaviour of a uranium tailings deposit, a preliminary investigation was carried out using field and laboratory test data from the Eldor Mines

Rabbit Lake Project, published in Gulf Minerals (1980, 1981) and from Geocon file records. Trial analyses using a range of parametric values based on, or guided by, these field and laboratory test data, were carried out and the results compared with actual behaviour observed in the field.

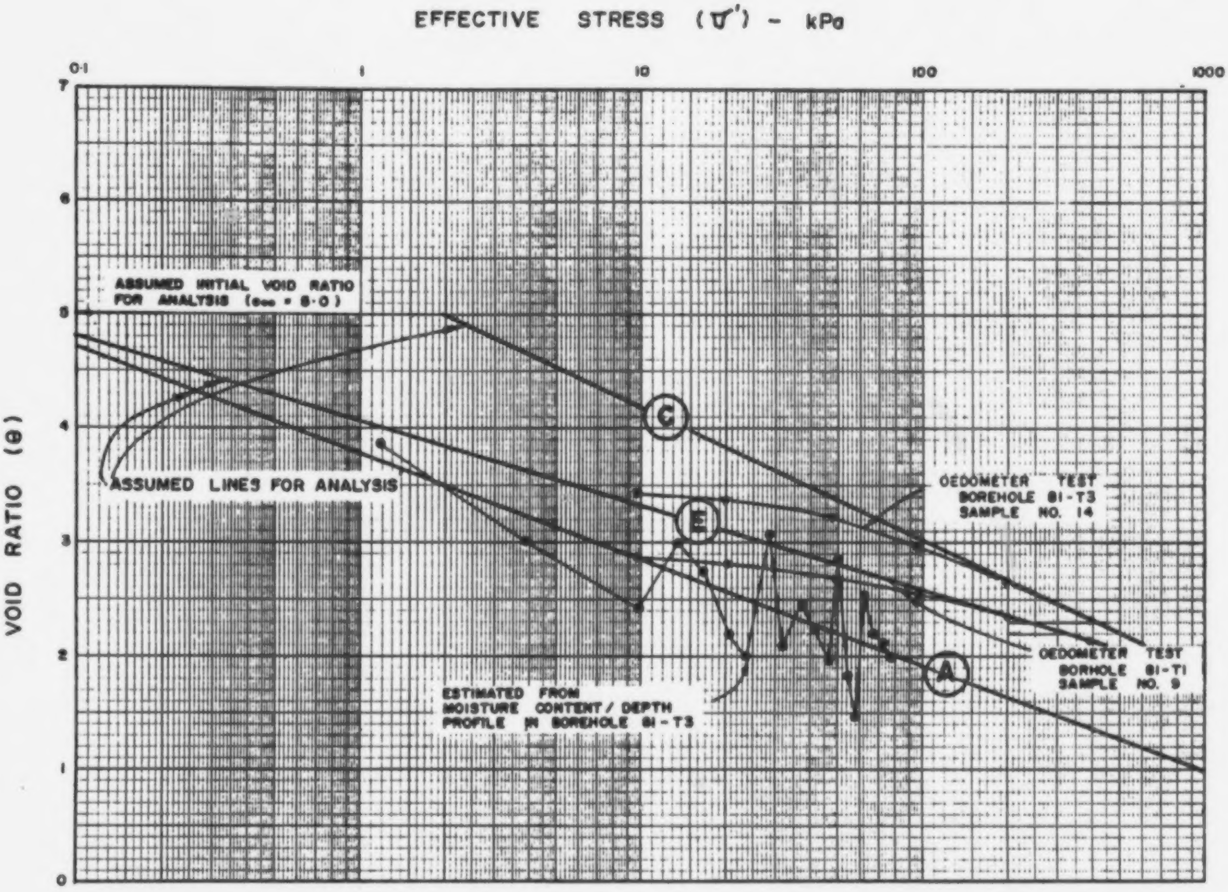
One of the main obstacles encountered in this part of the study which is to be carried out based on existing information only from the Canadian uranium mining industry, is that to-date, there has been no concerted effort in studying the consolidation behaviour of uranium tailings, particularly with reference to large strain consolidation and desiccation effects. It has been therefore necessary to exercise a considerable amount of discretion in assigning material characteristics for input to the analyses. In particular, with respect to evaporation/desiccation aspects, it was necessary to limit the study to sensitivity analyses covering a range of parametric values, due to the absence of reliable factual data.

Notwithstanding these limitations, it will be seen that the preliminary investigation presented in the following subsections reasonably establishes the validity of the modelling technique for the purposes intended, and highlights the important parameters which will be required for definitive studies in the future. Certain areas of improvement of the existing form of the computer program to enhance its capability in applications of this sort, are also suggested.

4.2.3.1 Consolidation During Deposition

A description of the site conditions and history of tailings deposition of the Rabbit Lake tailings area is given later in Section 5.0, with summarized field and laboratory test data from a geotechnical investigation within the tailings carried out in 1981. This information is used for modelling the consolidation behaviour of the tailings during active deposition, representing an accretion of 6 years of deposition operation.

Four boreholes were put down within the area as shown in Figure 5-1 in Section 5.1 of this report. Two oedometer tests carried out together with a void ratio-depth profile determined from samples taken from Borehole 81-T3 provide the principal source of information for use as input data for analyses discussed below. The results of these tests are shown in Figures 5-2, 5-3 and 5-4 in Section 5.1 of this report. Excess pore-water pressures measured in three piezometers installed close to Borehole 81-T3 during the investigation are also shown in Figure 5-2. This information, reprocessed into e- ' and e-k relationships for direct input into the analyses are shown in Figures 4-1 and 4-2. In-situ falling head permeability test results obtained in the boreholes are also shown in Figure 4-2. These tests however, reflect average permeabilities of the anistropic tailings which is anticipated to have significantly higher horizontal permeabilities than vertical permeabilities, whereas the oedometer tests yield vertical permeability values only. In one-dimensional consolidation modelling under consideration, vertical permeability is the required parameter for input. Therefore, little weight is given to the field permeability values except to indicate that the oedometer test results might have slightly under estimated the vertical permeability due to smearing effects during sample preparation. This evidence is based on the considerable separation of the field permeability data from the oedometer test data by some two to three orders of magnitude which is higher than th anticipated from the effect of anisotropy.



NOTE:

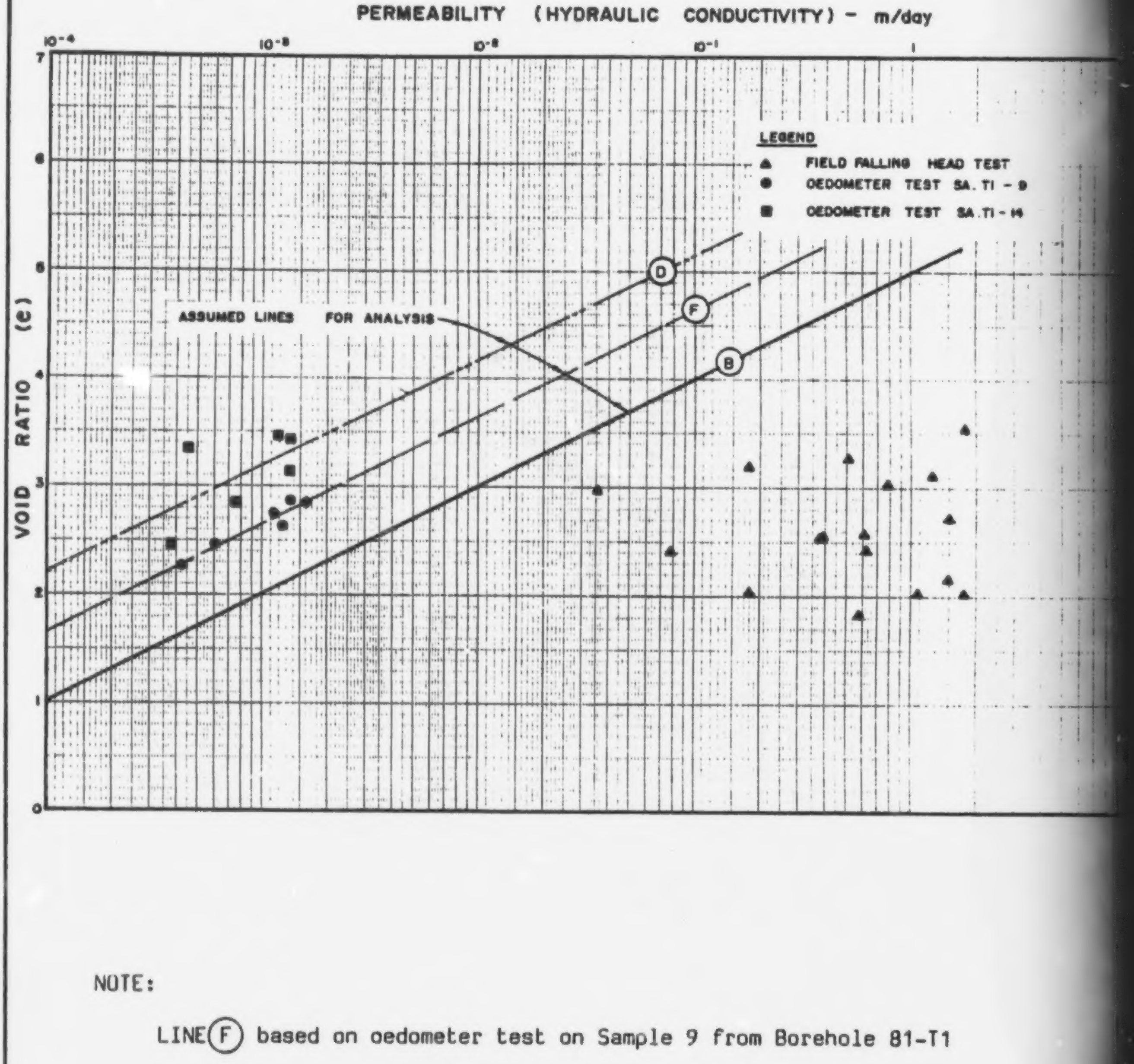
LINE Ⓐ based on moisture content/depth profile in Borehole 81-T3

LINE Ⓒ based on oedometer test on Sample No. 14 from Borehole 81-T3

LINE Ⓔ based on oedometer test on Sample No. 9 from Borehole 81-T1

FIGURE 4-I $e - \sigma'$ RELATIONSHIP FROM FIELD AND LABORATORY TEST DATA - RABBIT LAKE PROJECT

(Gulf Minerals,1981b)



NOTE:

LINE Ⓕ based on oedometer test on Sample 9 from Borehole 81-T1

LINE Ⓓ based on oedometer test on Sample 14 from Borehole 81-T1

LINE Ⓑ taken arbitrarily between oedometer test results and in-situ falling head test results, using a similar gradient as LINES Ⓓ and Ⓕ

FIGURE 4-2 e - k RELATIONSHIP FROM FIELD AND LABORATORY TEST DATA - RABBIT LAKE PROJECT
(Gulf Minerals, 1981b)

For input to the analyses, an average rate of accretion was computed from the total weight of material solids per unit area in plan represented by the column of tailings in Borehole 81-T3 accumulated over a 6-year period. The computed annual rate of accretion is 5.46 tonnes/m^2. The initial unconsolidated void ratio (e_{oo}) was assumed based on the observed void ratios in Borehole T1-3 near the surface of the tailings and the approximate correlation (Brown and Thompson, 1977):

$$w = 2.53\ LL_r$$

where

w = water content of sediment under zero stress
LL_r = liquid limit of dried and reconstituted soil

Based on the annual rate of accretion and initial void ratio indicated above, the Lagrangian (unconsolidated) thickness of tailings deposited annually is 4.25 m.

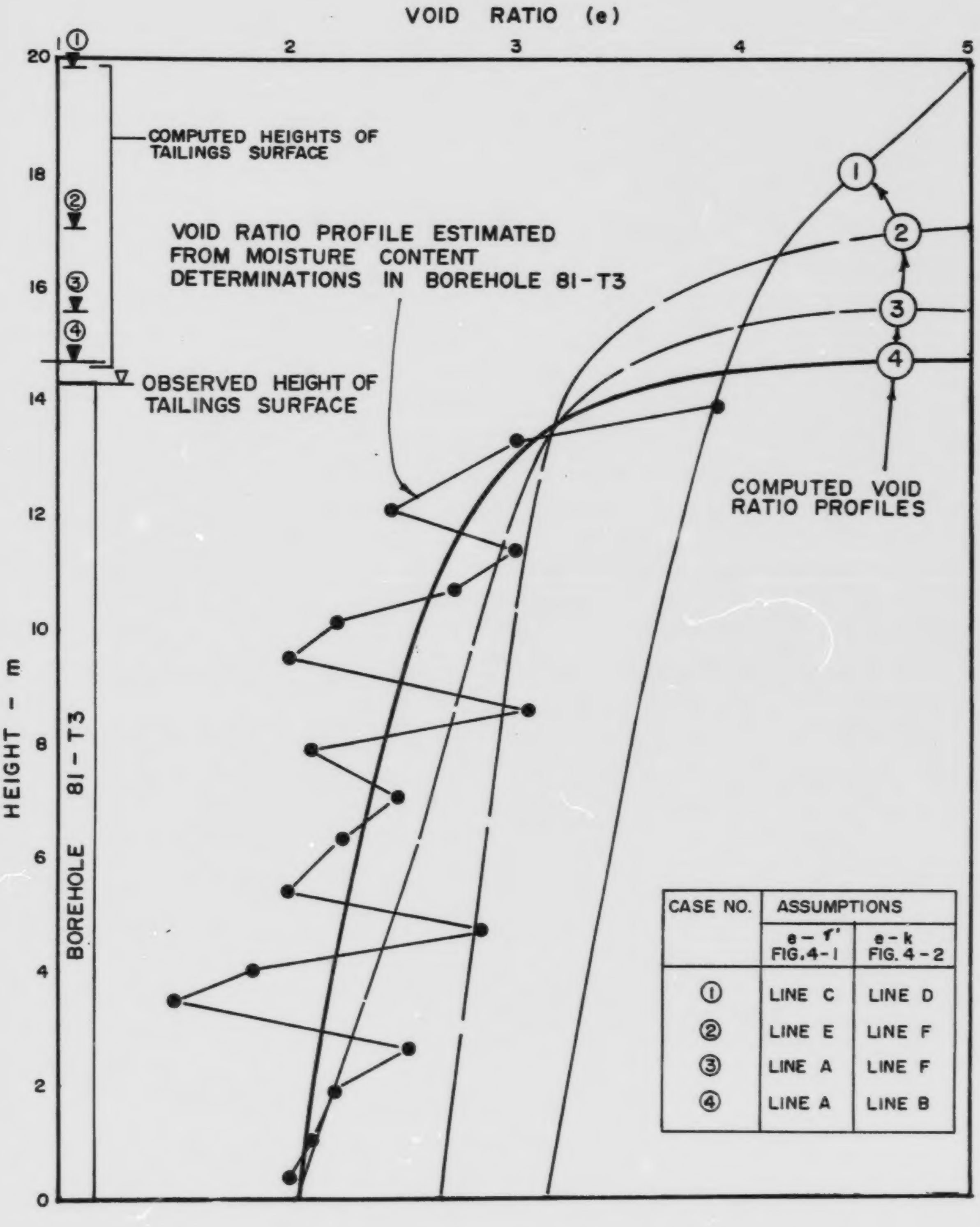
To simulate base drainage conditions, an average permeability of 0.864 m/day (10^{-4} cm/sec) for the foundation soil was used in the analyses with an assumed drainage path length of 5 m. This average permeability is reasonably representative of the natural overburden underlying the tailings at the site, which consists of thin surficial covers of muskeg and ablation till over a dense silty till.

Four cases were analysed using the above rate of accretion and lower boundary condition, combined with the following sets of e-σ' e-k relationships.

Case	e- ' (Fig. 4-1)		e-k (Fig. 4-2)	
	Line Designation	Origin of Data	Line Designation	Origin of Data
1	C	Oedometer test BH81-T3 Sample 14	D	Oedometer test BH81-T3 Sample 14
2	E	Oedometer test BH81-T1 Sample 9	F	Oedometer test BH81-T1 Sample 9
3	A	Field moisture content BH81-T3	F	Oedometer test BH81-T1 Sample 9
4	A	Field moisture content BH81-T3	B	Line F adjusted upward, considering in-situ falling head test data

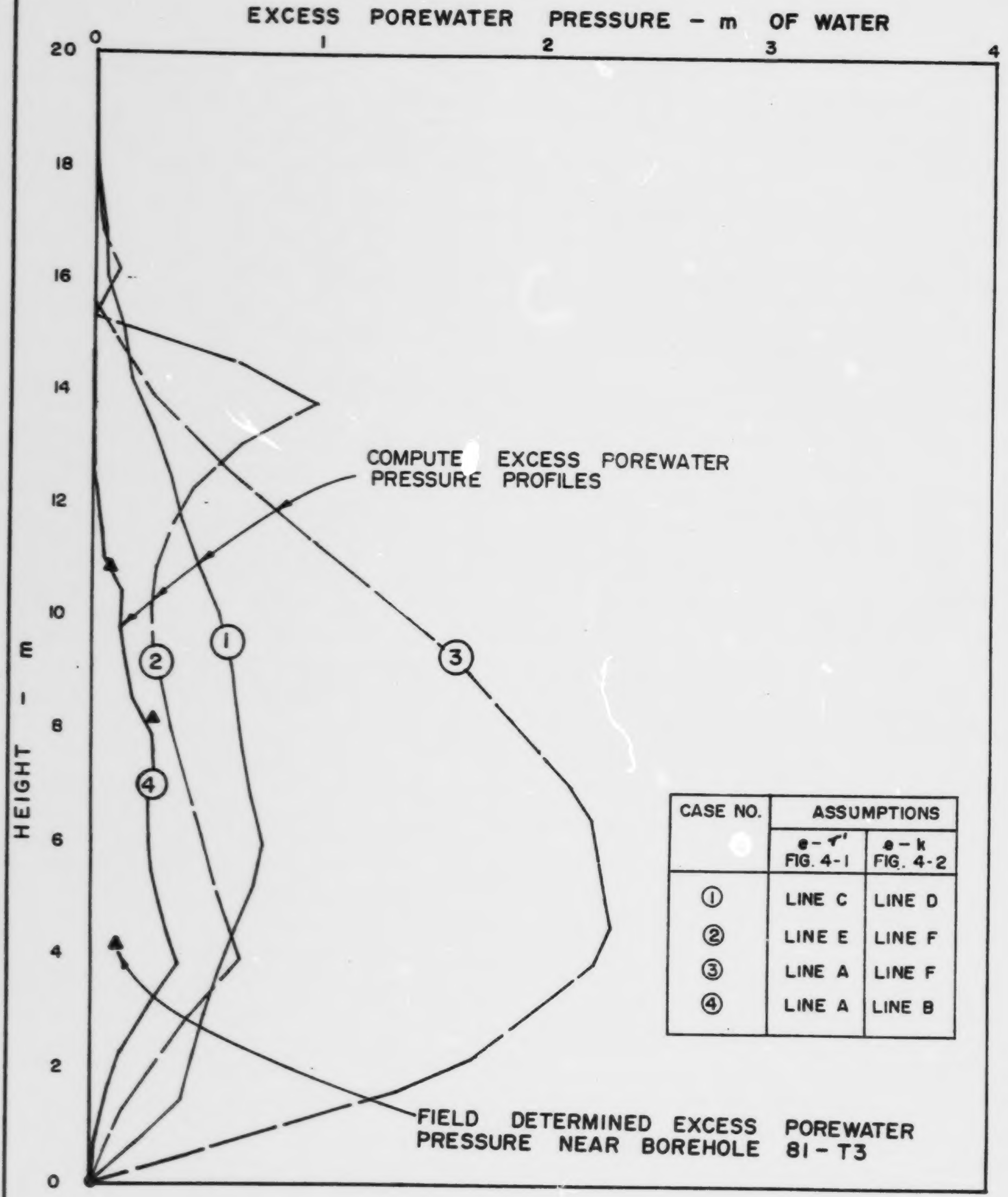
The results of the analyses are shown summarized in Figure 4-3 on the following page. From these results, it is clear that using the oedometer test results alone (Cases 1 and 2) gave very poor simulation of the field conditions. Both the actual height (convective height) of the tailings at the borehole location and the void ratio versus depth profile were significantly over-estimated. The computed excess pore-water pressures were also somewhat higher than those observed in the field from piezometers, as shown in Figure 4-4, although the differences are rather insignificant. Irregularities of the excess pore-water pressure profile in Case 2 could be due to slight instability in the numerical procedure of approximation resulting from the time step used in the finite difference mesh.

The discrepancies of the computed void ratio profiles and heights of tailings in Cases 1 and 2 clearly illustrate the difficulty in modelling the behaviour of a stratified deposit based on the results of small-scale laboratory tests alone. The significant variation in void ratio profiles obtained from consolidation characteristics of different samples from Borehole 81-T3, as shown



CASE NO.	ASSUMPTIONS	
	e – f' FIG. 4-1	e – k FIG. 4-2
①	LINE C	LINE D
②	LINE E	LINE F
③	LINE A	LINE F
④	LINE A	LINE B

FIGURE 4-3 COMPARISON OF COMPUTED AND FIELD DETERMINED VOID RATIO PROFILES RABBIT LAKE PROJECT



CASE NO.	ASSUMPTIONS	
	$e - \sigma'$ FIG. 4-1	$e - k$ FIG. 4-2
①	LINE C	LINE D
②	LINE E	LINE F
③	LINE A	LINE F
④	LINE A	LINE B

FIGURE 4-4 COMPARISON OF COMPUTED AND FIELD DETERMINED EXCESS POREWATER PRESSURE PROFILES

in Figure 4-3 (Case 1 and Case 2), verifies the anticipated stratified nature of the tailings deposit discussed earlier in Section 2.2. The computed profiles relative to the field data infer that the samples tested are representative of the finer-grained layers rather than the overall average properties of the tailings en-masse.

Following the above reasoning it was decided first to revise the e-σ' relationship derived from an average void ratio profile in the field, while keeping the e-k characteristic as in Case 2. The results of this analysis, shown as Case 3 in Figures 4-3 and 4-4, gave better simulation of the void ratio distribution and height of the tailings, but even higher excess porewater pressures compared to Case 2. It was of course illogical to revise the e-σ' characteristic to reflect, on the average, coarser-grained material without revising the e-k characteristic at the same time. That the average permeability characteristic in the field is probably higher than those obtained from the oedometer tests is also supported by evidence from the field falling head test results shown in Figure 4-2. This then led to the analysis shown as Case 4 in Figures 4-3 and 4-4, in which the permeability characteristic represented by line "B" was used. The results as shown produced excellent agreement with the field observations in every respect.

It can be concluded from the above that, provided representative input characteristics are used, the mathematical modelling by the program PCDDF will enable accurate prediction of large strain consolidation behaviour of uranium tailings under fully saturated conditions, to be made.

4.2.3.2 Comparison with Infinitesimal Strain Theory

To illustrate the superiority of the non-linear finite strain theory over the linear infinitesimal strain theory, a parallel analysis of the above problem was carried out by the method given in Gibson (1958) described earlier in 3.1.1.

The variable permeability and non-linear compressibility material properties assumed in the above analyses were converted into a range of c_v values used in the Gibson's method and the range of computed heights of the tailing and excess pore-water pressure profiles are shown in Figure 4-5 on the following page, for comparison with the solution in Case 4 described above. This shows that the infinitesimal strain theory significantly underestimated the rate of consolidation of the tailings.

4.2.3.3 Desiccation

Verification of the accuracy of the desiccation portion of the program is of course not possible at this time due to the lack of field data for comparison purposes. Given the empirical nature of this part of the program, however, it can be said that the accuracy of prediction would depend on the measurement and selection of parametric values input to the analysis.

4.3 Parametric Study

4.3.1 Saturated Self-weight Consolidation

Sensitivity of the analysis to input $e-\sigma'$ and e-k characteristics has already been discussed in Section 4.2.3.1 above. Further examination of the influence of the assumed rate of accretion on the analysis is discussed below.

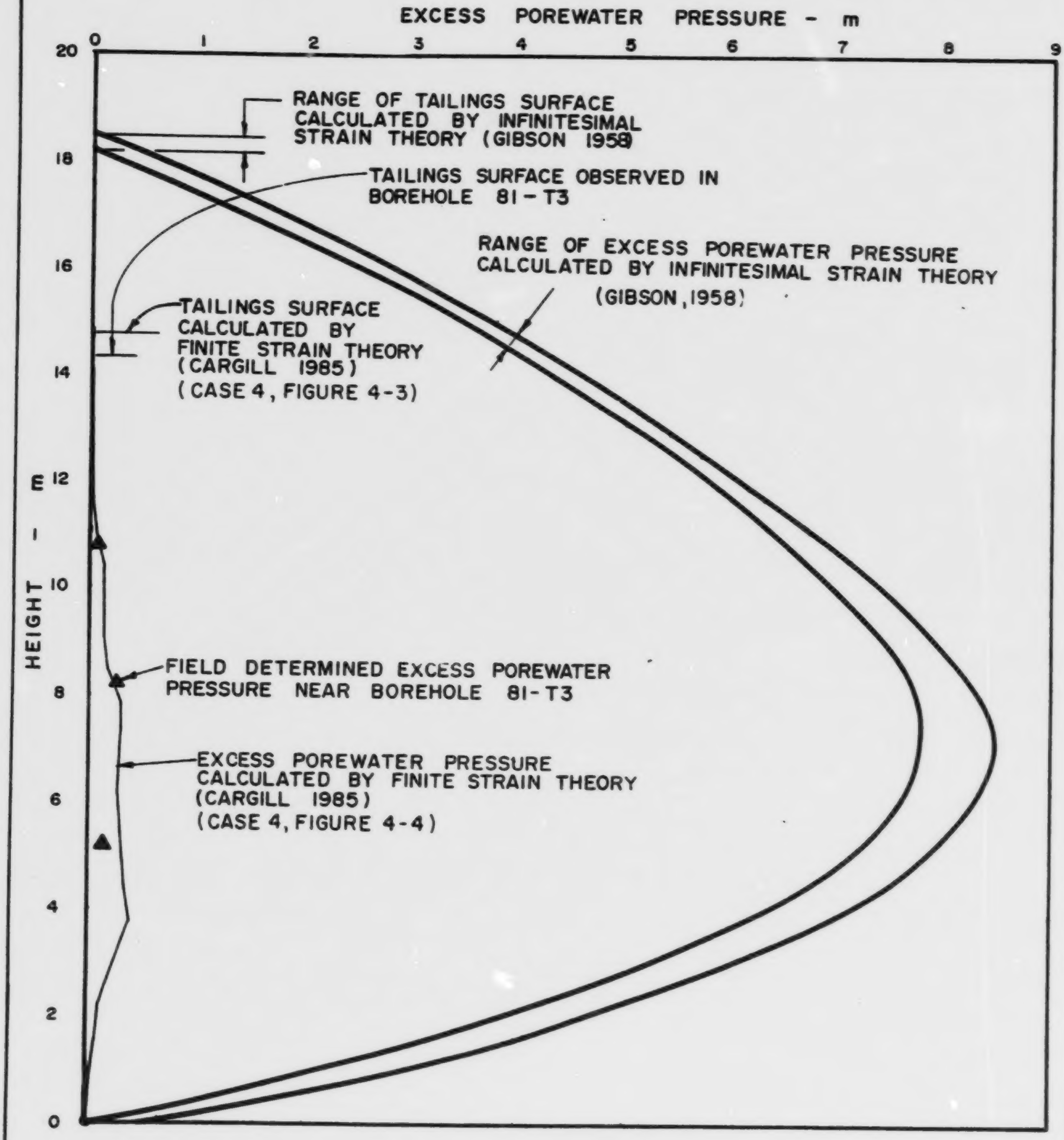


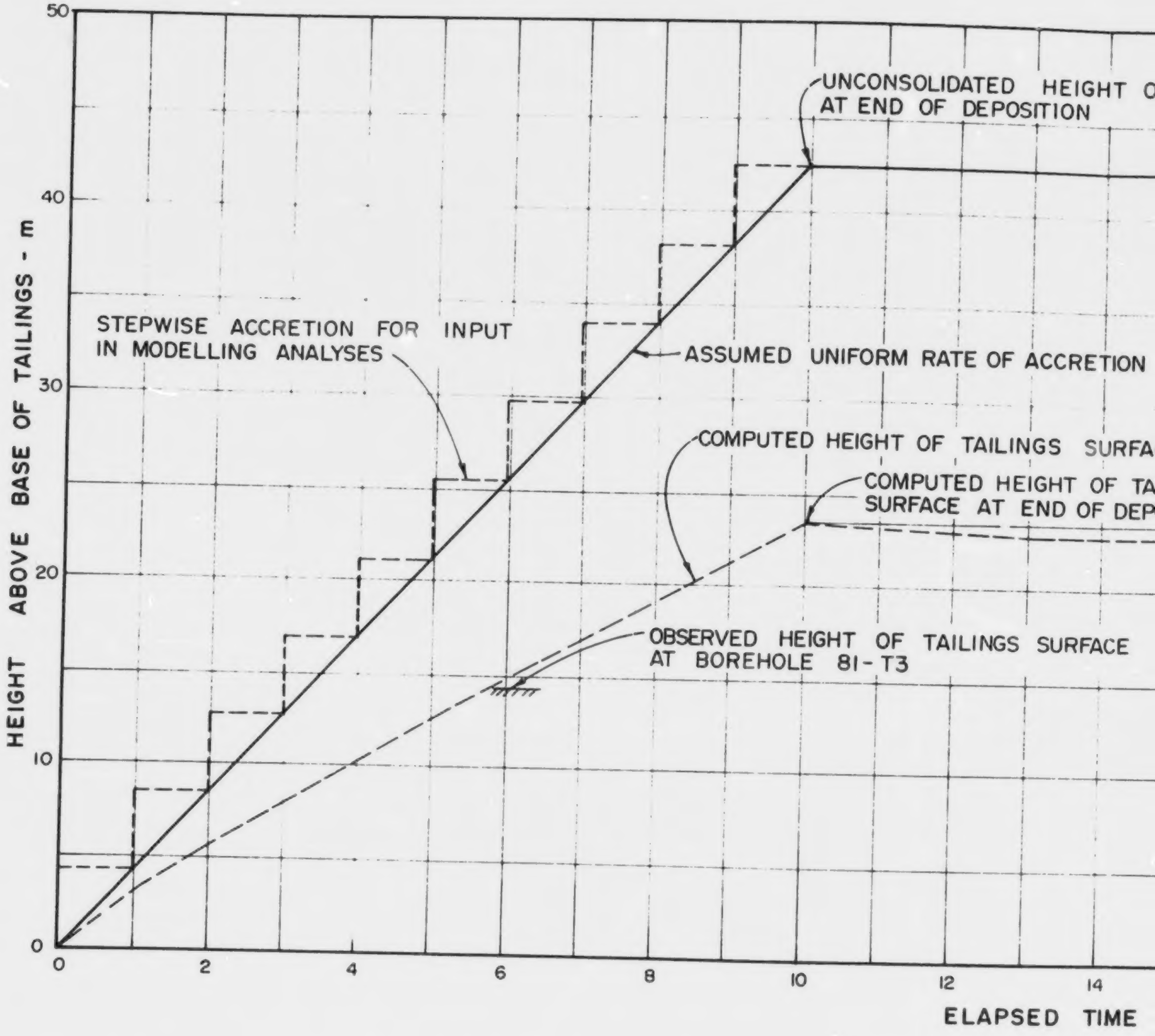
FIGURE 4-5 COMPARISON OF FINITE STRAIN AND INFINITESIMAL STRAIN CONSOLIDATION THEORIES

Deposition of tailings in the Rabbit Lake tailings area was essentially completed towards the end of 1985, representing a total of 10 years of accretion. The analysis described as Case 4 was therefore extended to cover a 10-year period and the results of the analysis are shown in Figure 4-6. The computed height of the tailings at the end of deposition is 23.35 m. The actual height of the tailings at the end of deposition in 1985 was not determined. However, preliminary information from drilling in the vicinity in Part B of this study indicated that the height of the tailings at the location may currently be about 2 m lower than the computed value. This discrepancy could be largely accounted for by the actual rate of accretion in the last four years which may be less than that assumed in the analysis. This is due to significant increases in the surface area of the tailings deposit in the later stages as its surface elevation rises, and is actually reflected in the decreasing heights of annual dam construction required in the last few years of operation.

The sensitivity of the computed height of the tailings to the assumed rate of accretion highlights the importance of the history of deposition to the accuracy of the analysis, and suggests the need to refine the model in Part B of this study.

The computed degree of consolidation settlement at the end of deposition is 97.9 percent with a residual primary settlement of 0.41 m. Extension of the analysis to cover a 10-year period after the end of deposition, assuming no desiccation to take place, indicates that this residual primary settlement would essentially be completed within five years from the end of deposition. This computed rate of residual settlement is also shown in Figure 4-6.

Although the computed convective height of the tailings surface is sensitive to the assumed Lagrangian rate of accretion, the degree of consolidation computed at the end of deposition is relatively unaffected by minor



UNCONSOLIDATED HEIGHT O
AT END OF DEPOSITION
STEPWISE ACCRETION FOR INPUT
IN MODELLING ANALYSES
ASSUMED UNIFORM RATE OF ACCRETION
COMPUTED HEIGHT OF TAILINGS SURFA
COMPUTED HEIGHT OF TA
SURFACE AT END OF DEP
OBSERVED HEIGHT OF TAILINGS SURFACE
AT BOREHOLE 81-T3
HEIGHT ABOVE BASE OF TAILINGS - m
0
10
20
30
40
50
0
2
4
6
8
10
12
14
ELAPSED TIME

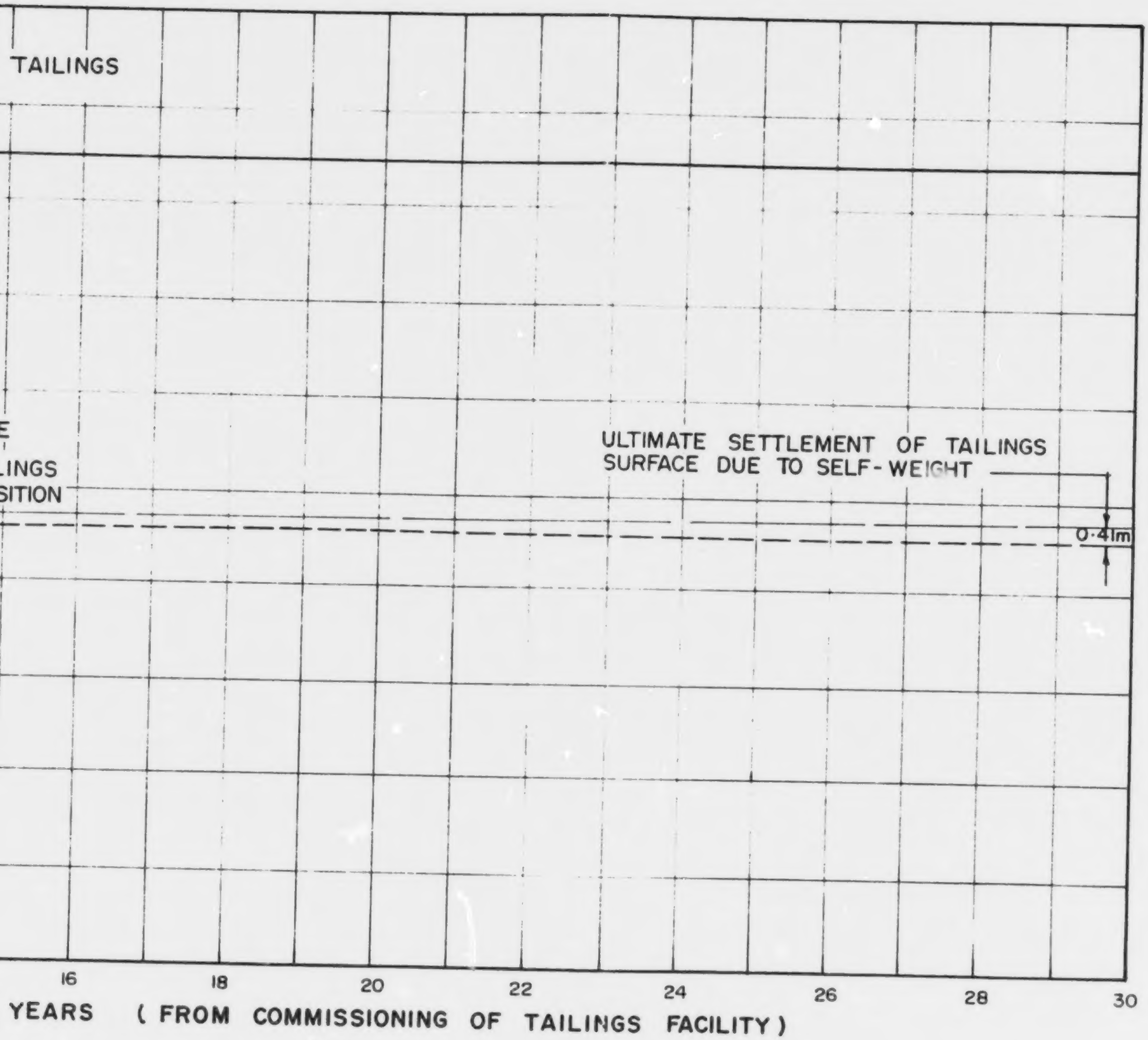


FIGURE 4-6
MODELLING OF ACCRETION AND SELF-WEIGHT CONSOLIDATION - RABBIT LAKE PROJECT

changes in the rate of accretion. The magnitude of an error in the predicted convective height due to inaccuracy in the assumed rate of accretion, of course, should not be construed as the magnitude of an error in the estimated settlement of the tailings surface. Rather, the percentage errors in those quantities bear a more direct relationship. Thus the probable error in the computed residual settlement given above may be in the order of 10 percent or 0.04 m although the computed height of the tailings surface at the end of depositon may involve an error of about 2 m. It can be concluded therefore that computed post-deposition settlements are relatively insensitive to minor inaccuracy in the assumed rate of accretion during deposition.

4.3.2 Desiccation

The input parameters required to model the effects of desiccation are:

- the saturation limit, e_{sL}
- the desiccation limit, e_{DL}
- the maximum depth of second stage drying or depth of water level, $H_2(max)$
- the gross degree of saturation of the desiccated crust, S
- meteorological statistics in the form of monthly precipitation and class A pan evaporation rates
- drainage and evaporation efficiencies.

The first three parameters have dramatic impact on the rate and magnitude of post-deposition settlements. Because of the empirical nature of these parameters, and the present state of knowledge related to the desiccation behaviour of uranium tailings, accurate prediction of long term settlements due to evaporation settlement is not possible. For the purpose of this part of the study, however, it was considered important to demonstrate the sensitivity of long term settlement to these parameters. A series of analyses were therefore carried out using a wide range of assumed parametric values and the results are discussed below.

The first set of values for e_{sL} and e_{DL} were selected based on the general guideline given in Cargill (1985):

$$e_{sL} = 1.8\,\rho_s(LL)$$
$$e_{DL} = 1.2\,\rho_s(PL)$$

where

ρ_s = specific gravity of solids
LL = liquid limit
PL = plastic limit

using average properties of the Rabbit Lake tailings given in Section 5.1 later in this report for guidance, an initial set of values:

$$e_{sL} = 3.59$$
$$e_{DL} = 1.71$$

were selected. Because these values appeared to be high judging from the states of desiccation they were intended to represent, two other sets of lower values were assumed in the sensitivity study. These are:

$$e_{sL} = 2.00\ ;\ 1.26$$
$$e_{DL} = 0.60\ ;\ 0.60$$

Combined with each set of limit void ratios, a series of assumed maximum depth of Stage 2 drying ranging between 0.5 m and 5.0 m were used in the analyses. In each case, the gross degree of saturation assumed was 0.75, the drainage efficiency was 1.00 and the evaporation efficiency during Stage 1 drying was 1.0, decreasing linearly with lowering of the water level in Stage 2 drying to zero at the assumed limit of depth of drying. Monthly evaporation rates at the Rabbit Lake site given in Section 5.1 were used throughout.

A summary table of the cases analysed is given below with reference to the accompanying Figures where the results of the analyses are shown.

SERIES NO.	CASE NO.	ASSUMED VALUES				FIGURE NO.
		e_{sL}	e_{DL}	H_2(max)		
1	5A	3.59	1.71	0.5 m		4-7A
	5B			1.0 m		4-7B
	5C			2.0 m		4-7C
	5D			5.0 m		4-7D
2	6A	2.00	0.60	0.5 m		4-8A
	6B			1.0 m		4-8B
	6C			2.0 m		4-8C
	6D			5.0 m		4-8D
3	7A	1.26	0.60	0.5 m		4-9A
	7B			1.0 m		4-9B
	7C			2.0 m		4-9C
	7D			5.0 m		4-9D

The results of these analyses clearly demonstrate the profound effect of desiccation on long term settlements. The influence of the depth of

(Text continues on page 4-32)

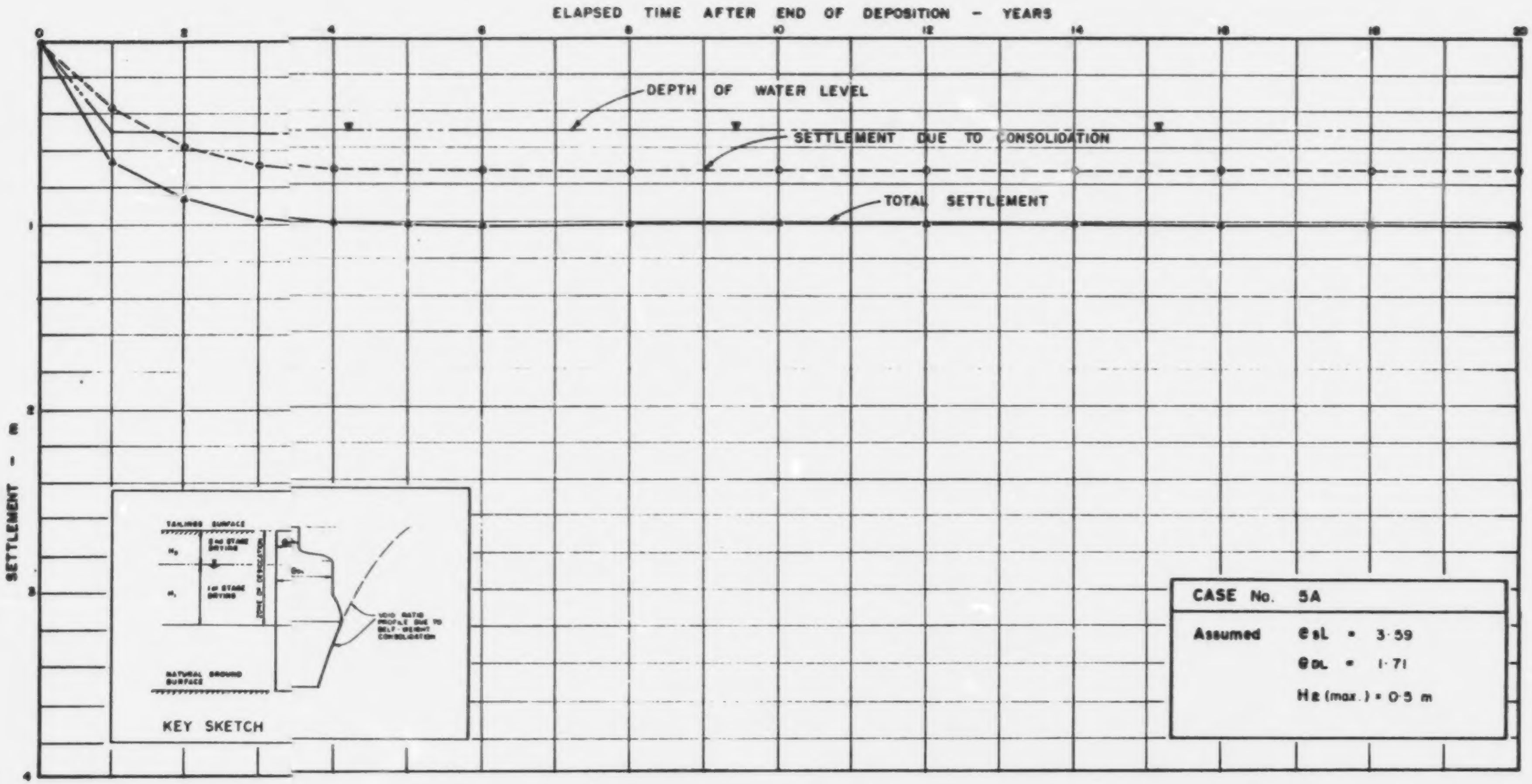


FIGURE 4-7A

SELF-WEIGHT CONSOLIDATION
RABBIT LAKE PROJECT

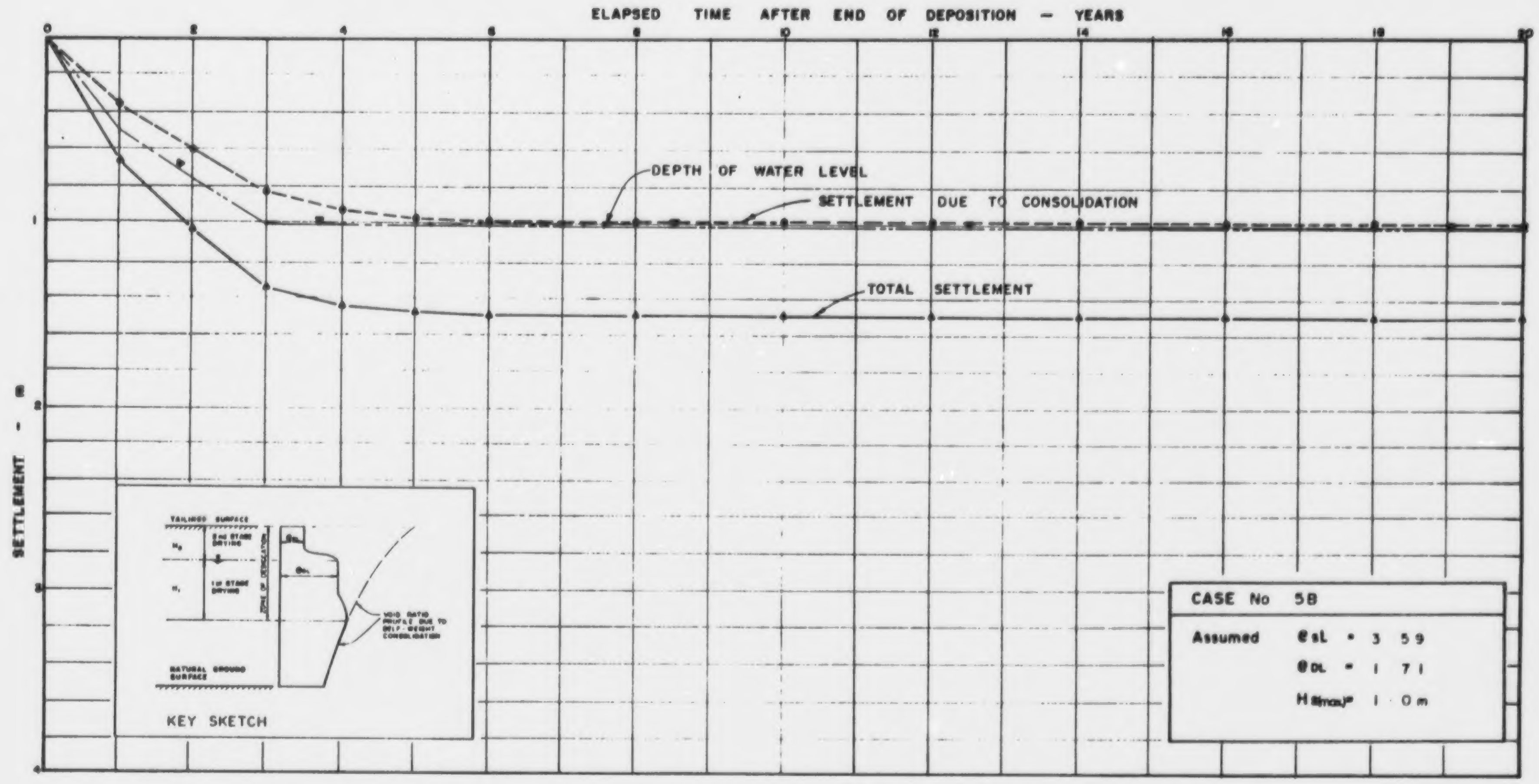


FIGURE 4-7B

SELF-WEIGHT CONSOLIDATION
RABBIT LAKE PROJECT

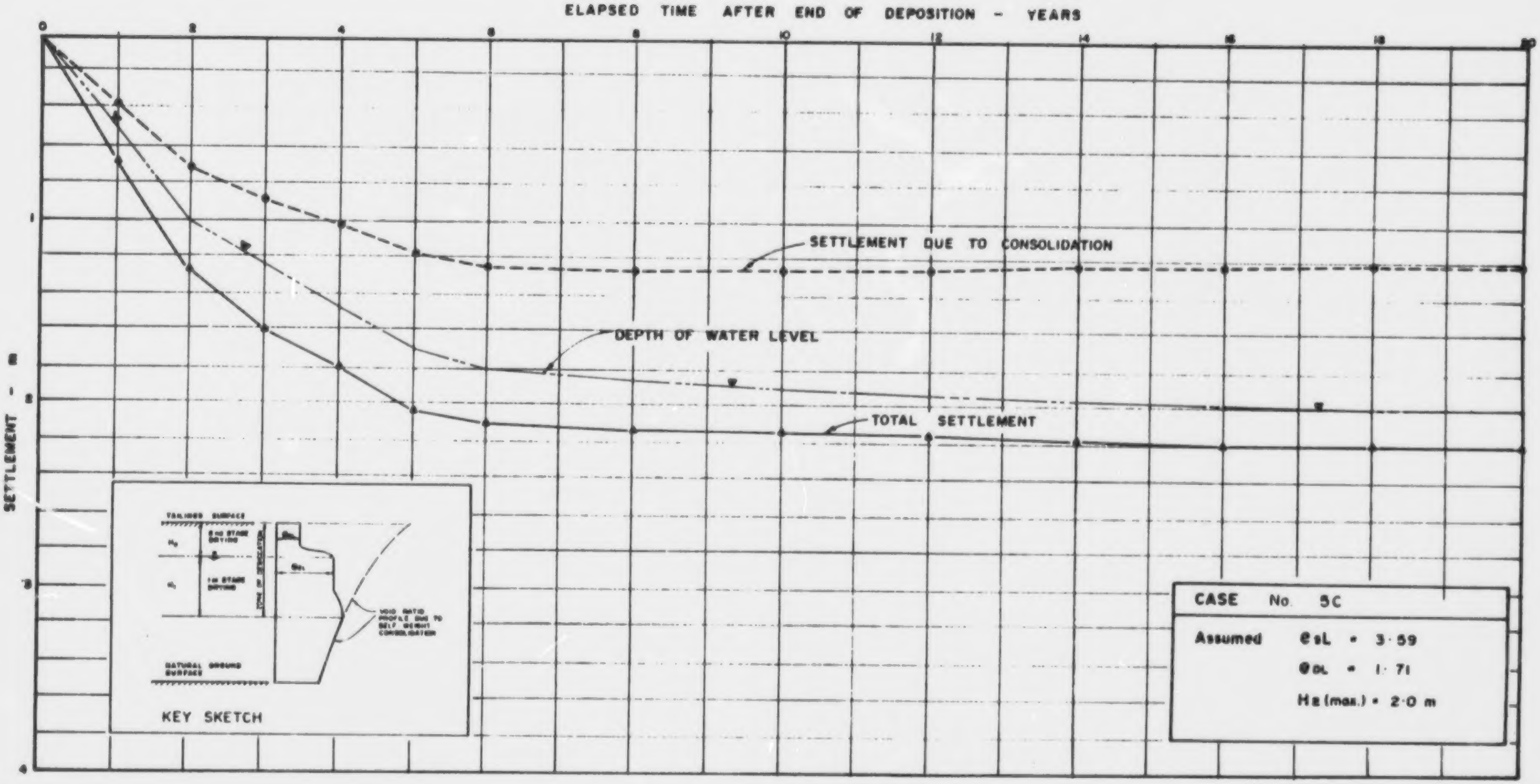


FIGURE 4 - 7C

SELF-WEIGHT CONSOLIDATION
RABBIT LAKE PROJECT

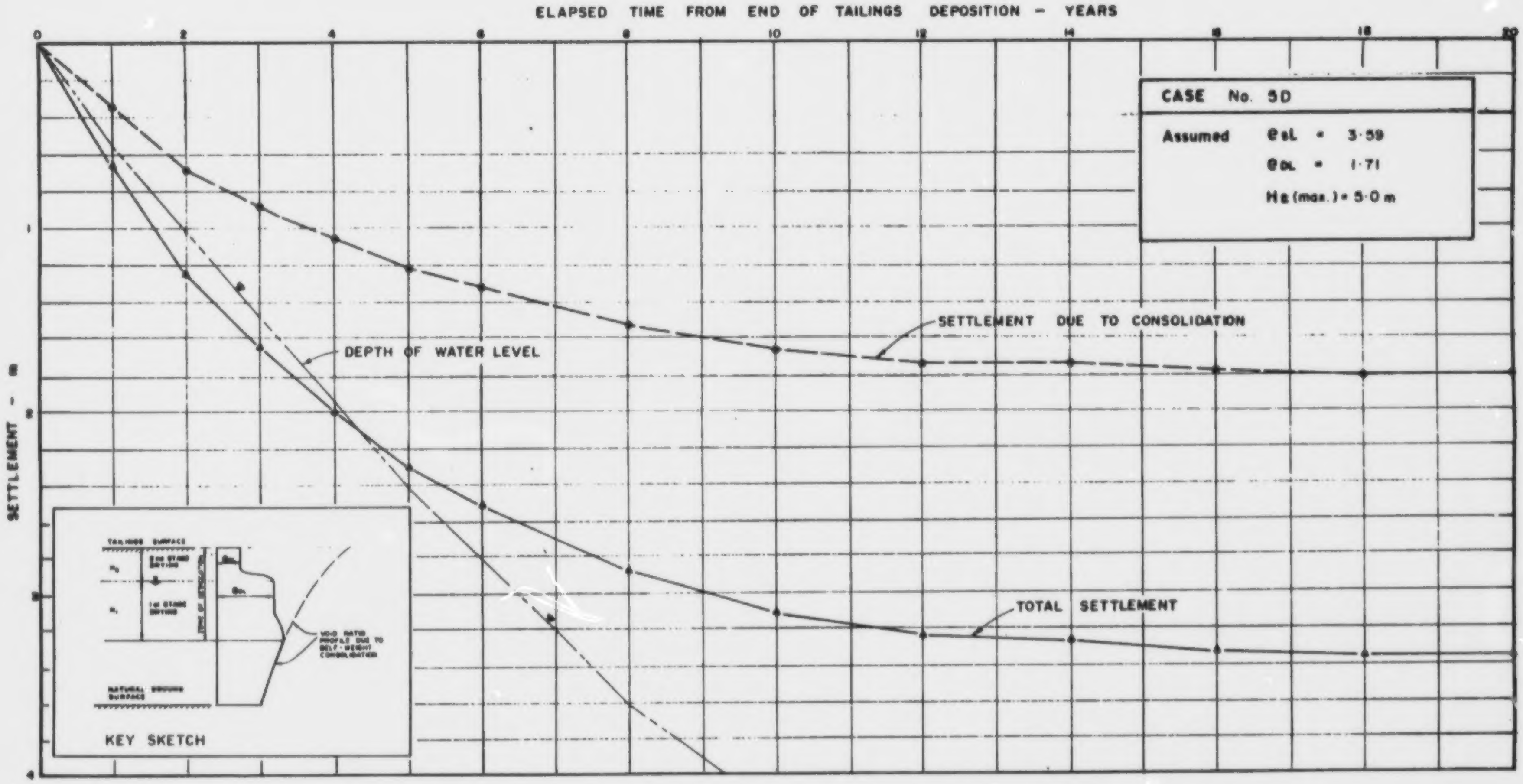


FIGURE 4-7D

SELF-WEIGHT CONSOLIDATION
RABBIT LAKE PROJECT

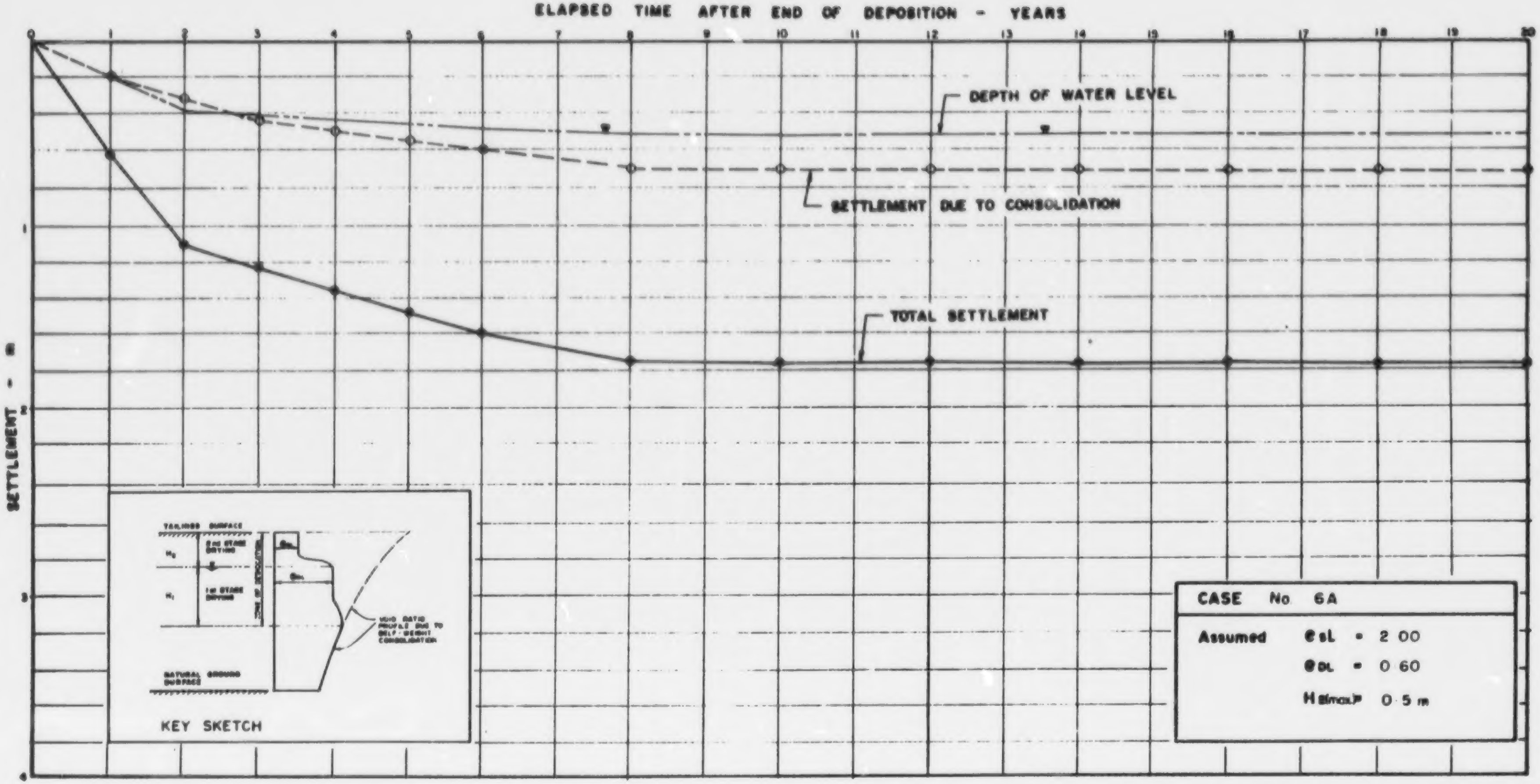


FIGURE 4 - 8A

SELF-WEIGHT CONSOLIDATION
RABBIT LAKE PROJECT

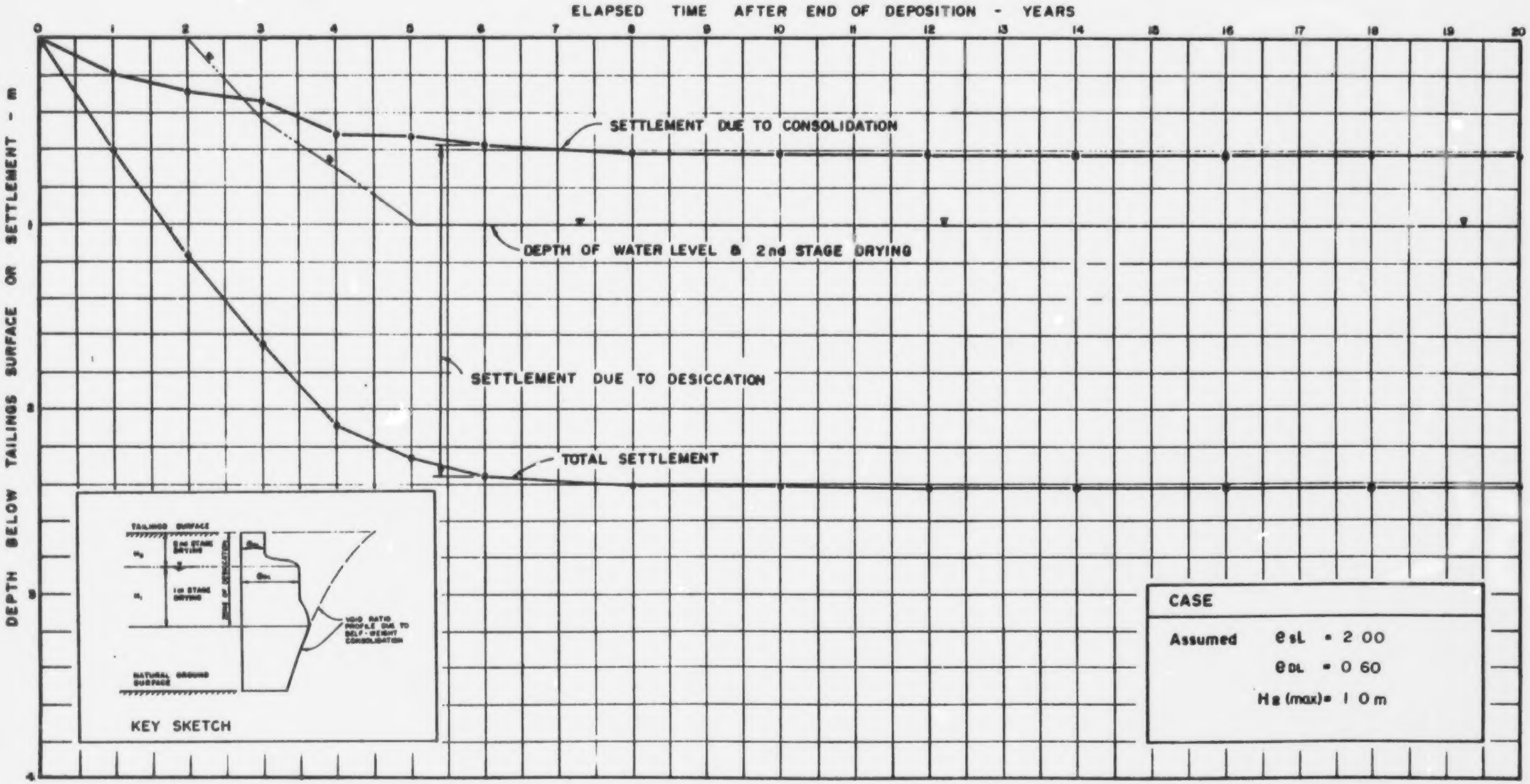


FIGURE 4-8B

SELF-WEIGHT CONSOLIDATION
RABBIT LAKE PROJECT

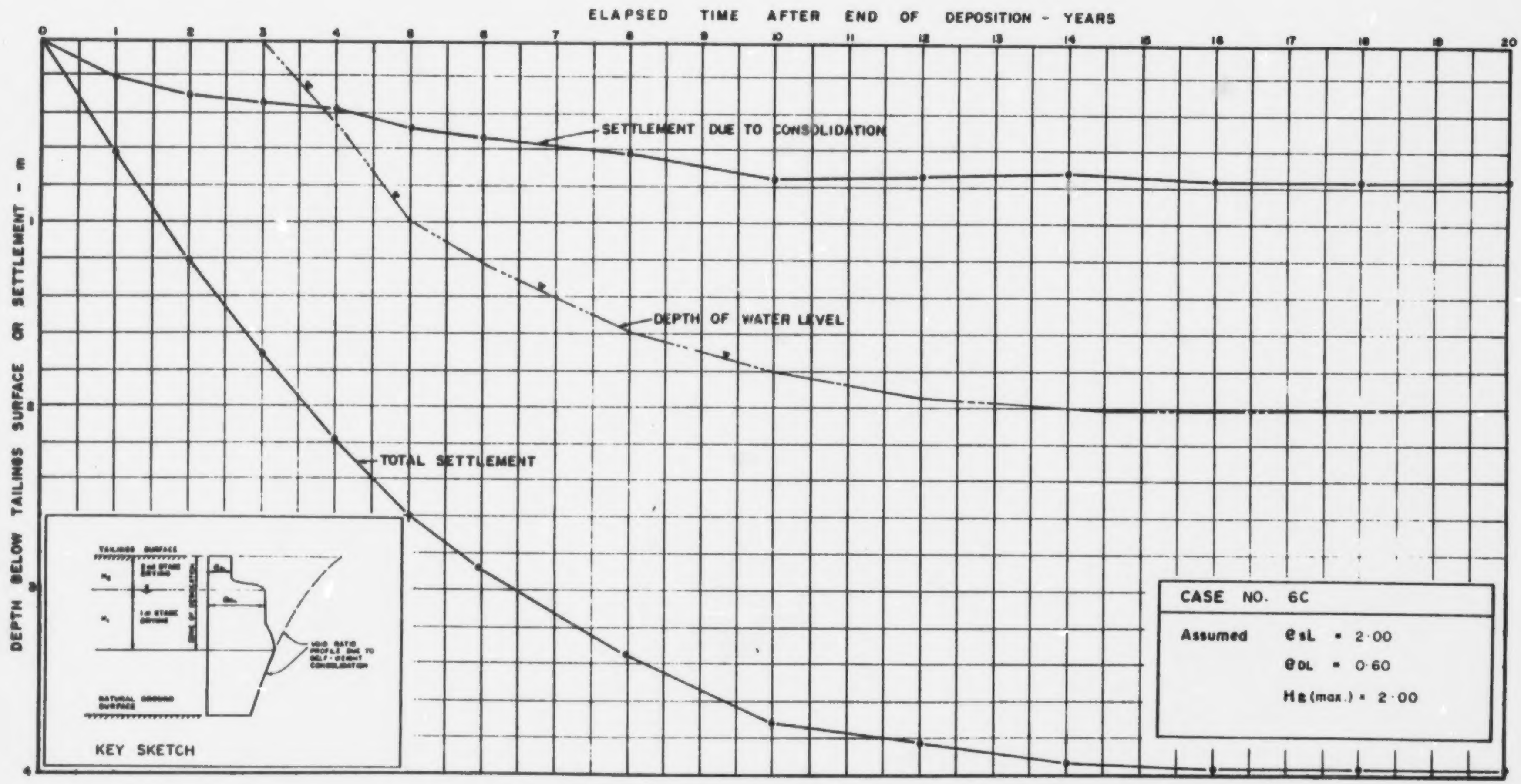


FIGURE 4 - 8C

SELF-WEIGHT CONSOLIDATION
RABBIT LAKE PROJECT

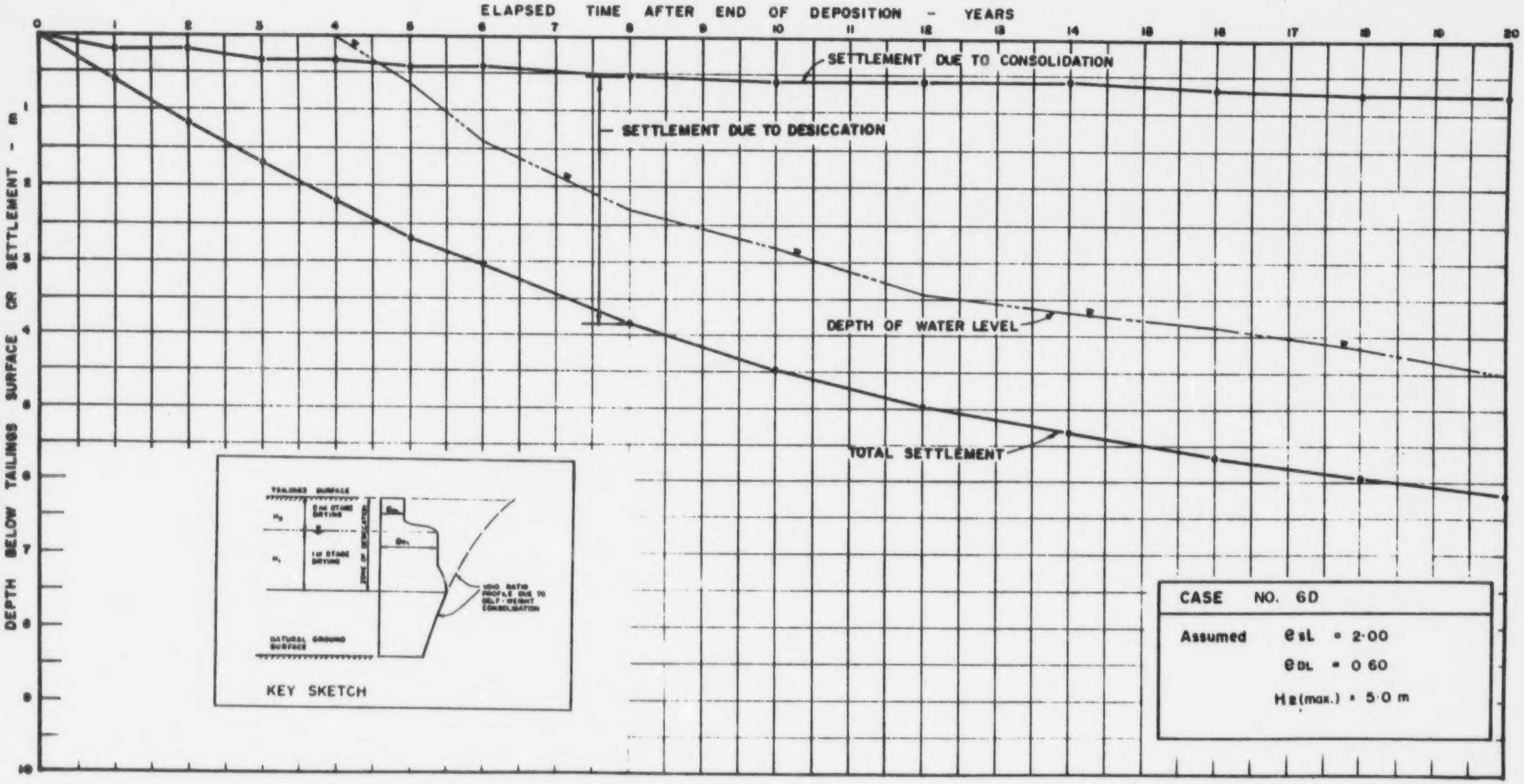


FIGURE 4-8D

SELF-WEIGHT CONSOLIDATION
RABBIT LAKE PROJECT

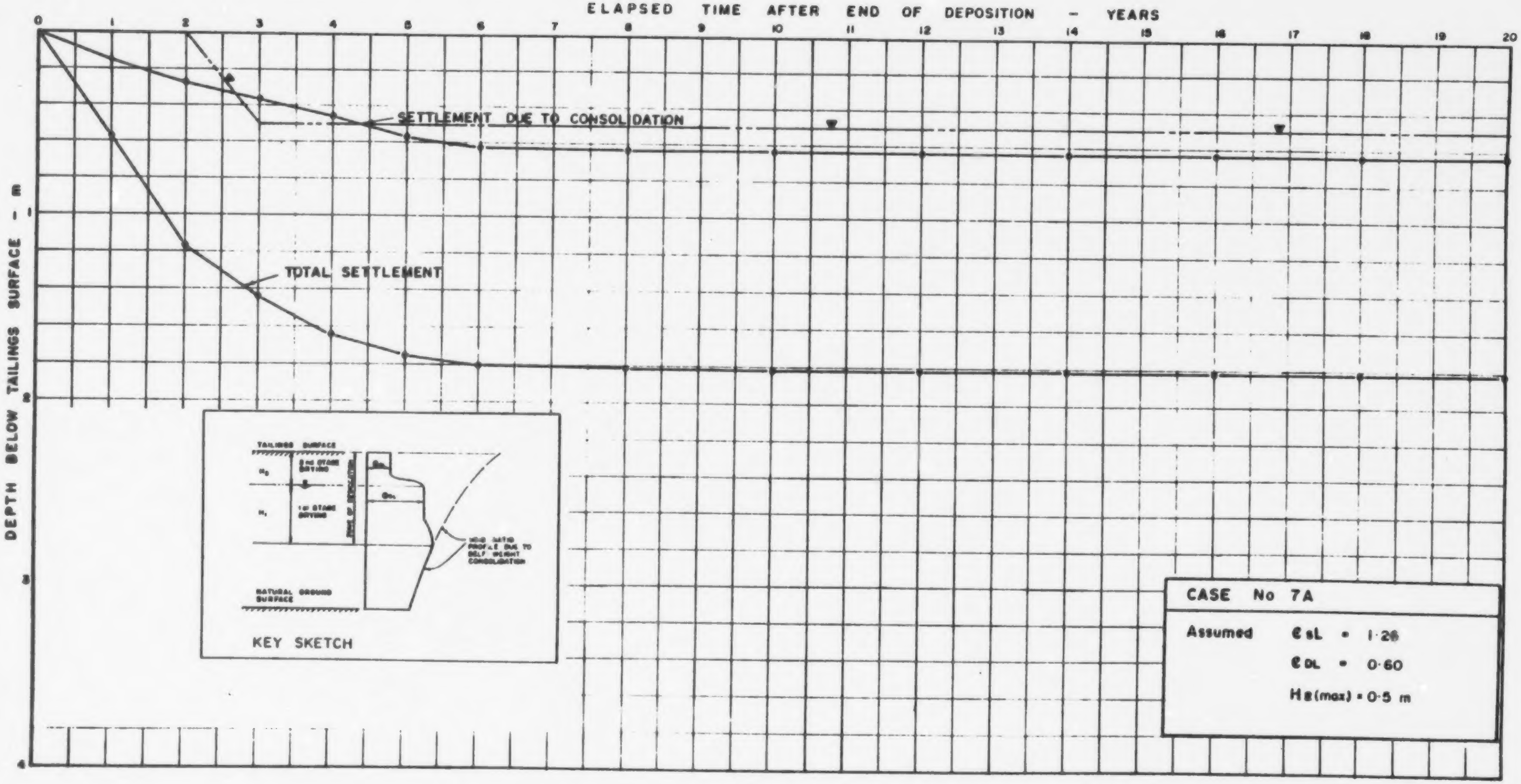


FIGURE 4 - 9A

SELF-WEIGHT CONSOLIDATION
RABBIT LAKE PROJECT

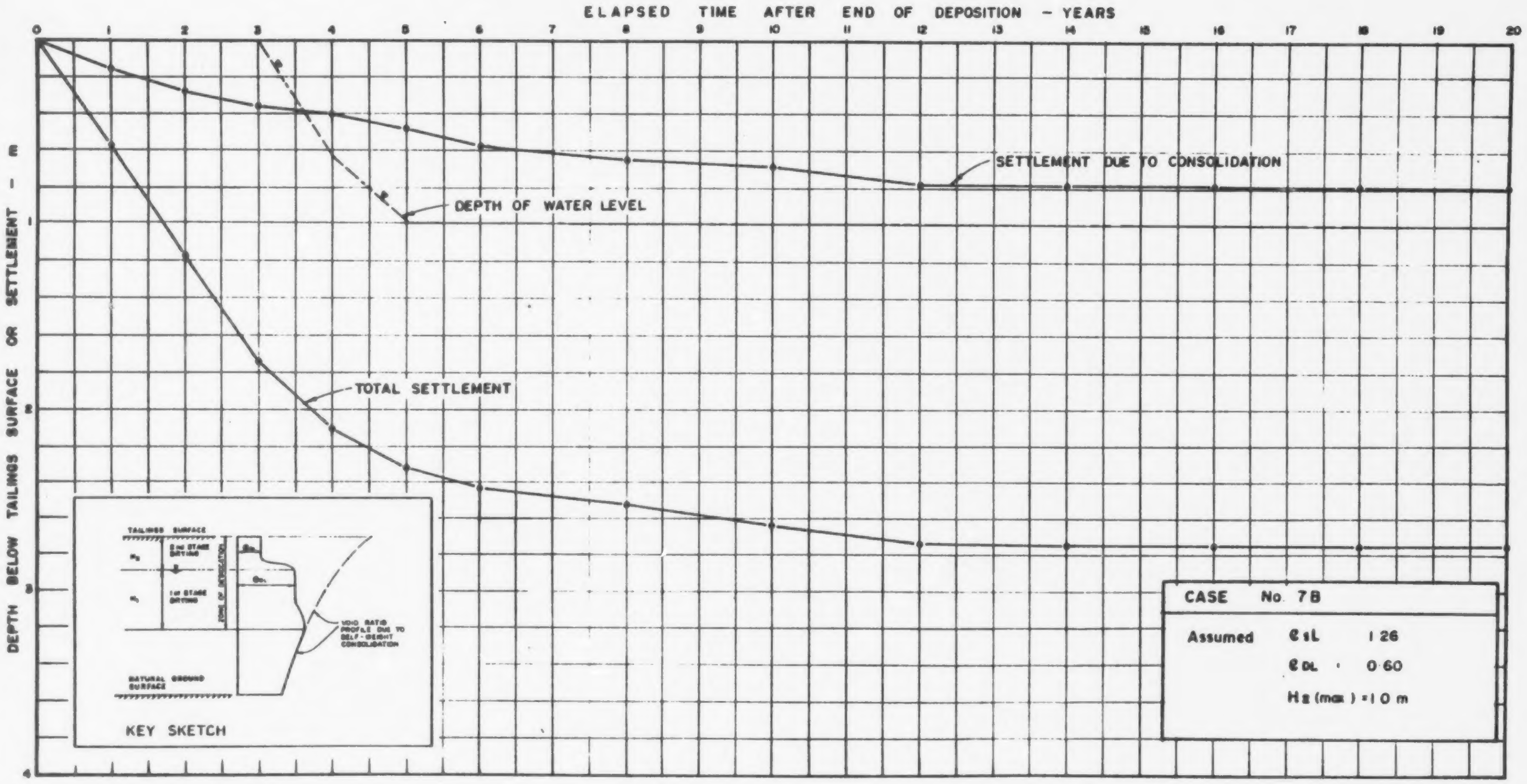


FIGURE 4-9B

SELF-WEIGHT CONSOLIDATION
RABBIT LAKE PROJECT

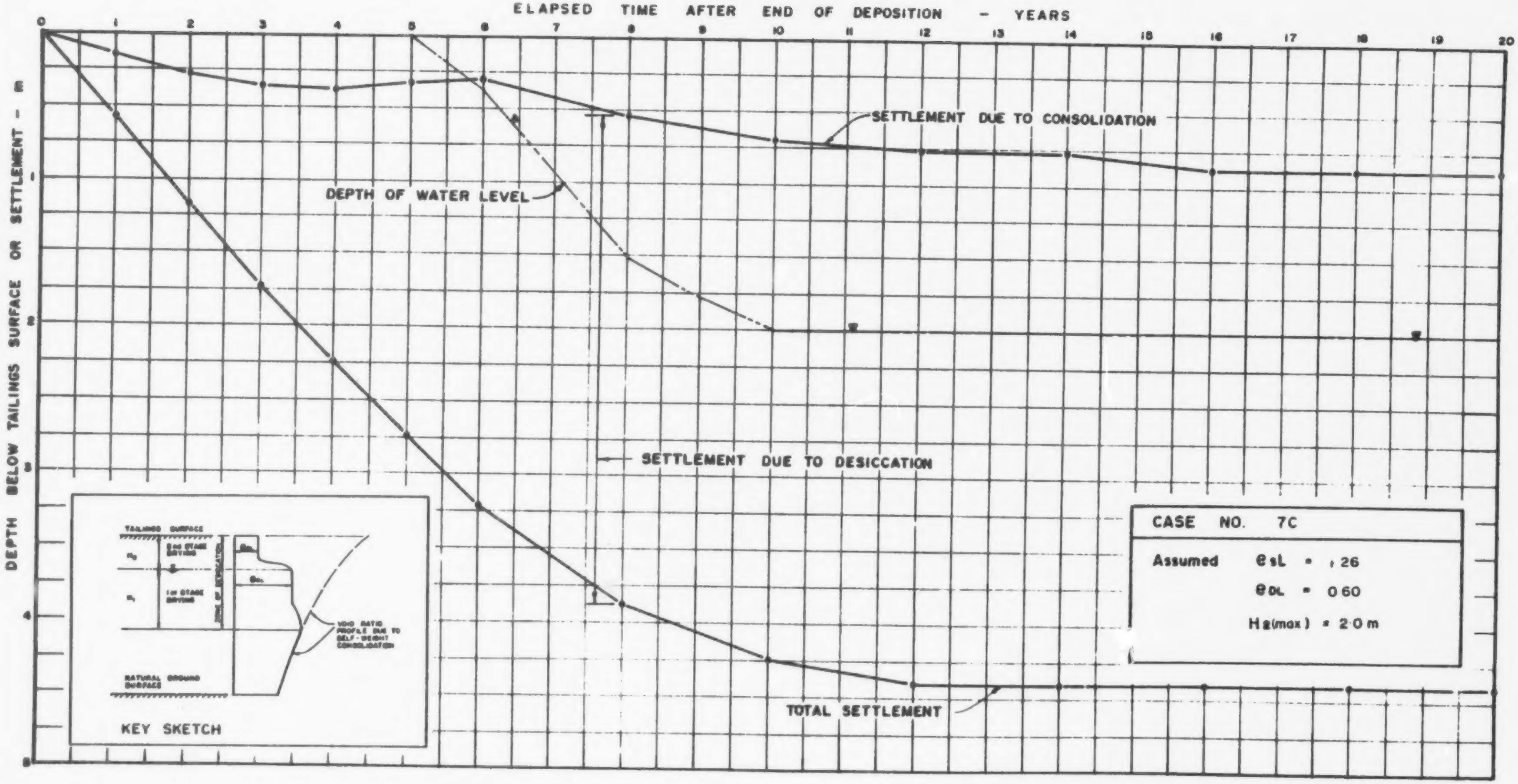


FIGURE 4 - 9C

SELF-WEIGHT CONSOLIDATION
RABBIT LAKE PROJECT

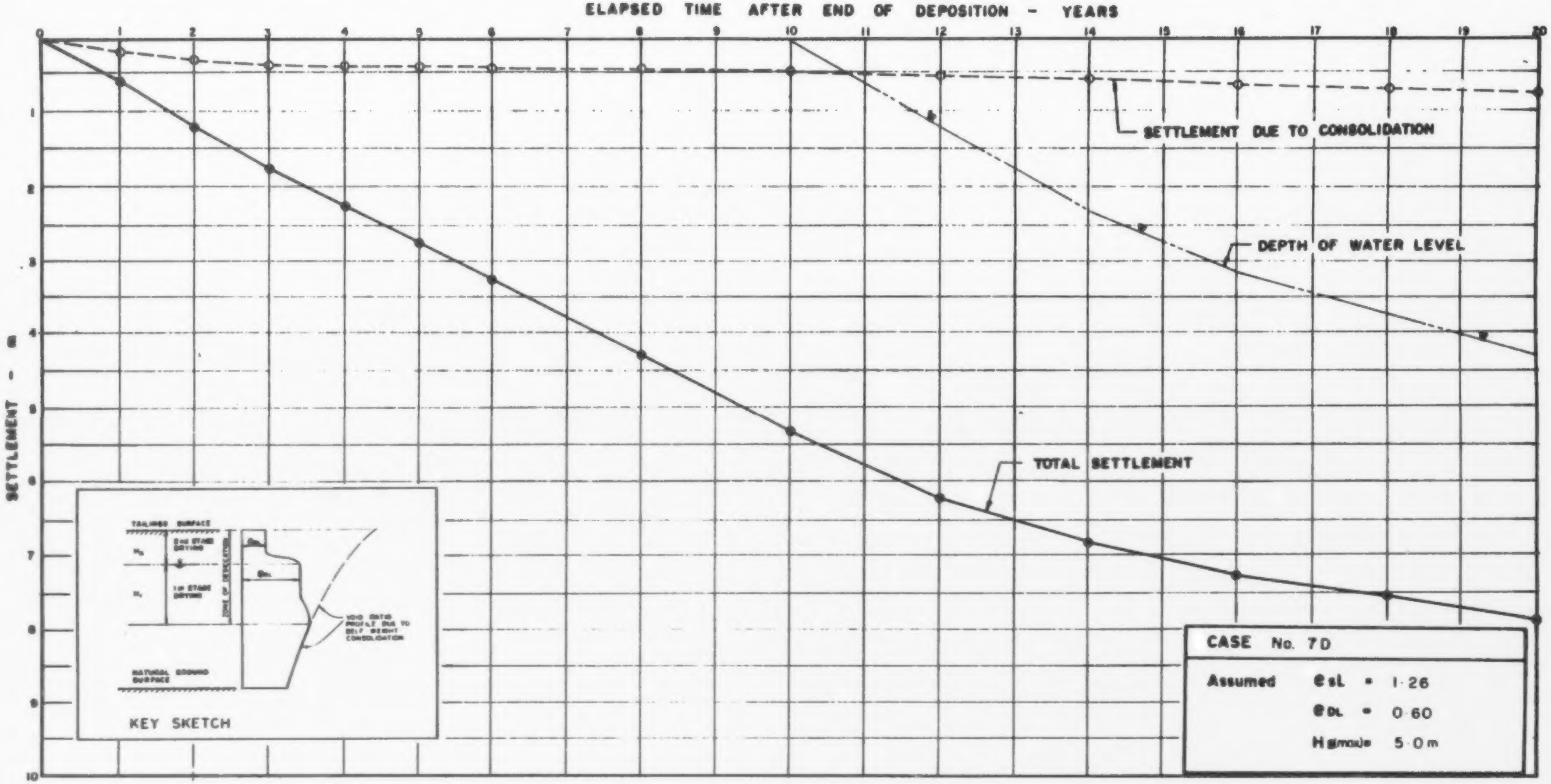


FIGURE 4 - 9D

SELF-WEIGHT CONSOLIDATION
RABBIT LAKE PROJECT

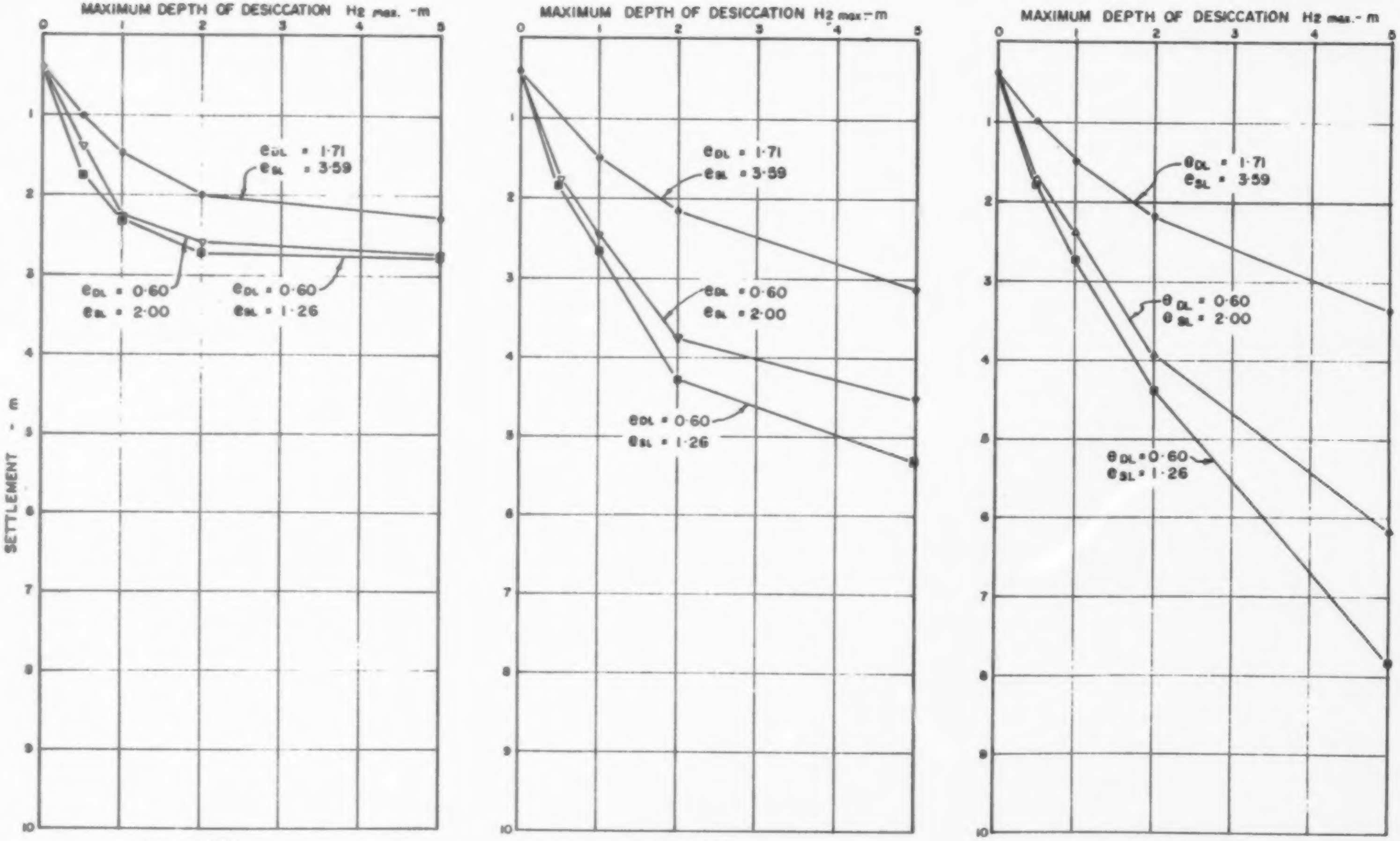
desiccation on both the duration and magnitude of settlements is particularly prominent. For instance, Figures 4-7A, 4-8A and 4-9A show that if the depth of desiccation is limited to 0.5 m, settlement would continue over a period varying between 5 and 8 years after end of deposition, whereas Figures 4-7D, 4-8D and 4-9D show that for a maximum depth of desiccation of 5.0 m, ongoing settlement could occur over periods of 18 years or more. The corresponding extremes of ultimate settlements range between about 1 m and well over 8 m.

To assist in comparing the various cases and thus the relative sensitivity of settlement to various parameters, a summary plot of computed settlements at selected time intervals is shown in Figure 4-10 on the following page.

4.4 Simulation of Cover

As indicated earlier, due to input limitations the settlements due to the placement of a cover on the surface of a tailings deposit, cannot be computed directly by the program PCDDF. With the onset of desiccation due to evaporation, this problem becomes even more complex due to the unknown effect of the cover as a barrier to evaporation from the tailings.

Without revising the program, a task which is beyond the terms of reference of the present study, the only means by which settlements due to a cover or surcharge loading could be computed is by simulating it with an equivalent layer of unconsolidated tailings. Evaporation cannot be allowed to take place in this case because it will result in increasing equivalent surcharge loading on the underlying tailings as the water level recedes below the surface of the simulated cover. With these restrictions, the only case that can be simulated directly is the one where a cover which acts as a perfect barrier to evaporation from the underlying tailings, is placed before any desiccation of the tailings occurs. Even in this case, some



# 5 YEARS

# 10 YEARS

# 20 YEARS

# ELAPSED TIME AFTER END OF DEPOSITION

FIGURE 4 - 10

SUMMARY PLOT OF COMPUTED SETTLEMENTS
-RABBIT LAKE PROJECT

errors are introduced due to the difference between the impedances of the simulated cover and the real cover to upward drainage of the consolidating tailings below. Such errors, however, can be ignored in practice. This method of simulation therefore gives an approximate estimate of the lower bound values of cover settlements.

The upper limit of cover settlements is given by the case where the cover is assumed to have no moderating effect on the rate of evaporation of the underlying tailings. To simulate this case it is necessary to first assume that the desiccated crust of the tailings is relatively incompressible and thus any contribution to cover settlement due to cover loading on the desiccated crust can be ignored. Then the following procedure can be used to estimate the cover settlement under the condition of unimpeded evaporation through the cover.

(1) Analyse three separate cases as follows:
    (a) no cover, no evaporation
    (b) no cover, full evaporation
    (c) with simulated cover, no evaporation
(2) Trace the depth of the desiccated crust with time in Case (b)
(3) Take the difference in settlements between cases (c) and (a) for points at the base of the desiccated crust at various times, determined in step (2)
(4) Add the settlement due to cover loading on the tailings below the desiccated crust determined in (3) to the corresponding total settlement of the tailings surface in case (b) to give the total settlement of the cover.

Using this method, a number of cases were analysed to estimate the post-construction settlements under various assumptions as shown in Figures 4-11A to 4-12C on the following pages.

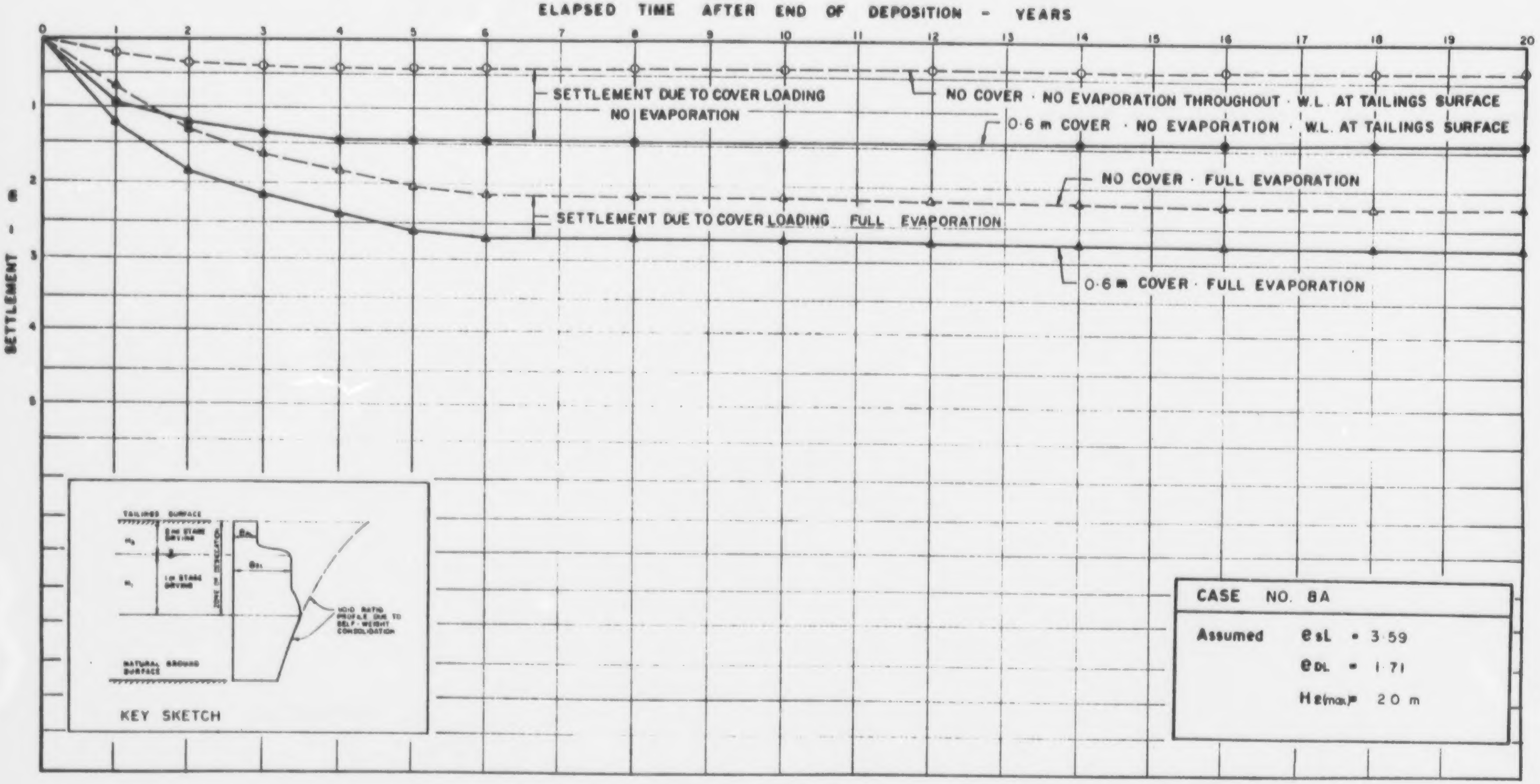


FIGURE 4 - 11 A

SETTLEMENT OF COVER CONSTRUCTED IMMEDIATELY AFTER END OF TAILINGS DEPOSITION

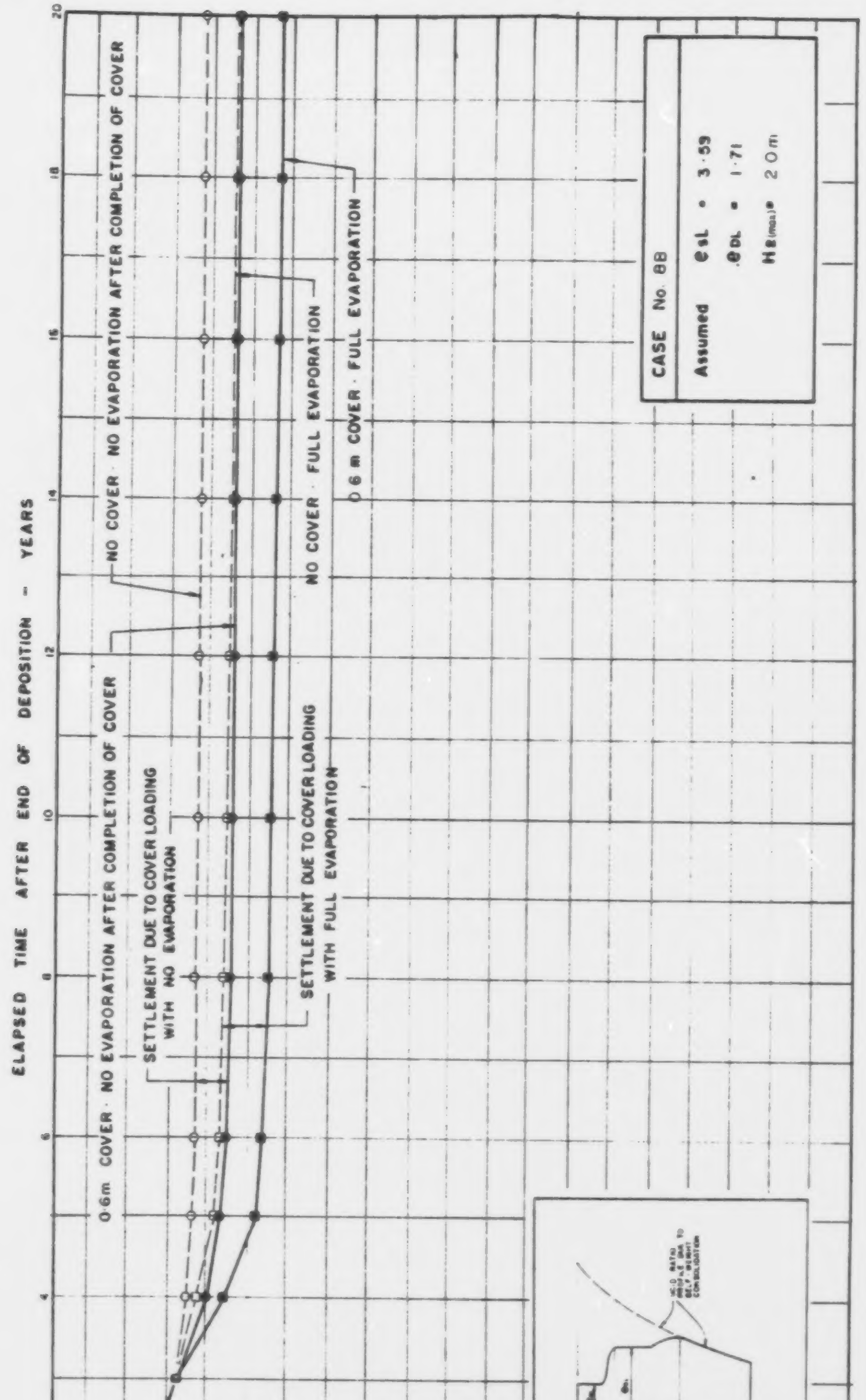


FIGURE 4 - 11 B

SETTLEMENT OF COVER CONSTRUCTED
3 YEARS AFTER END OF TAILINGS
DEPOSITION

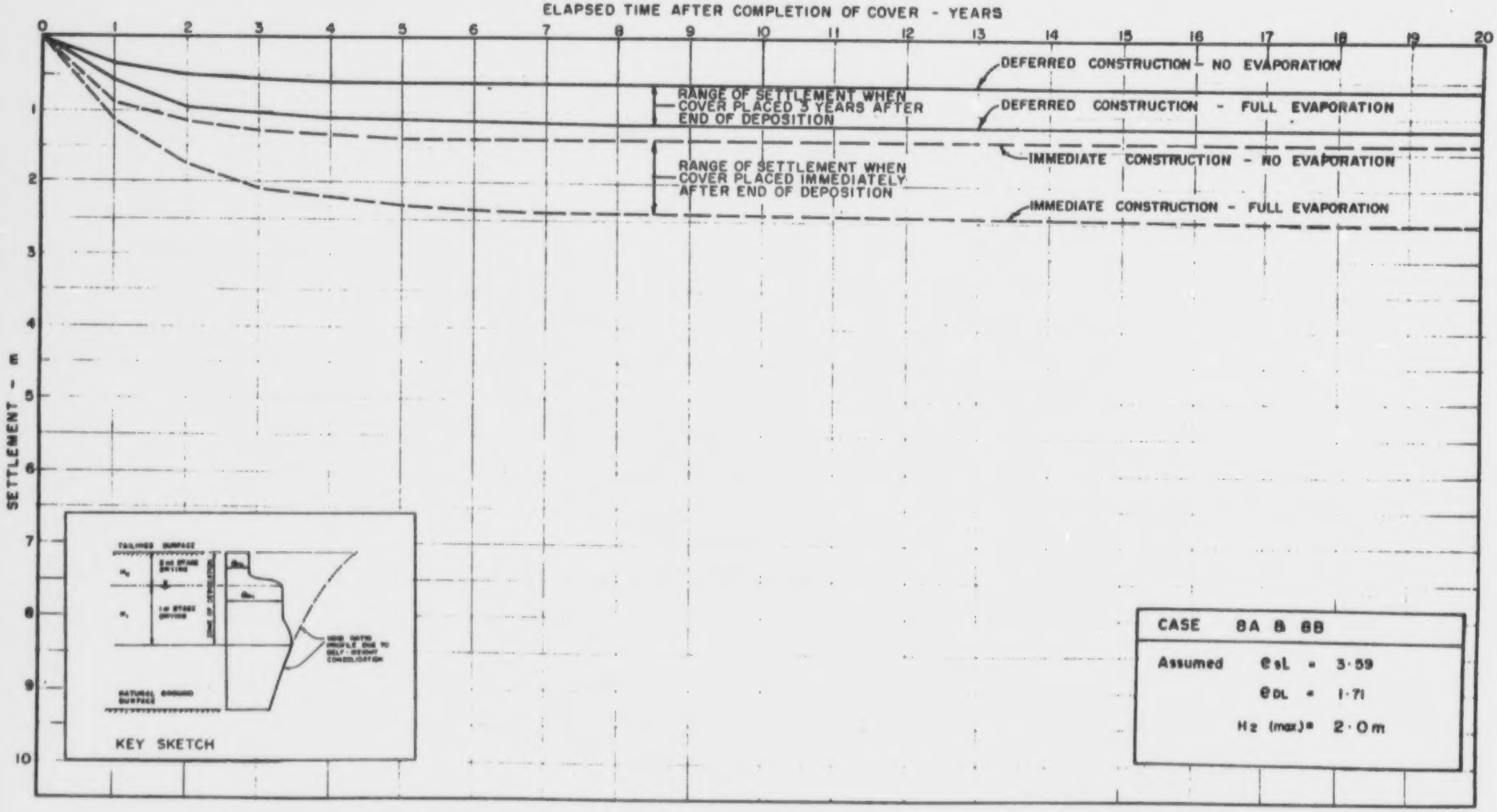


FIGURE 4 - 11C

REDUCTION OF COVER SETTLEMENTS
BY DEFERRING COVER CONSTRUCTION

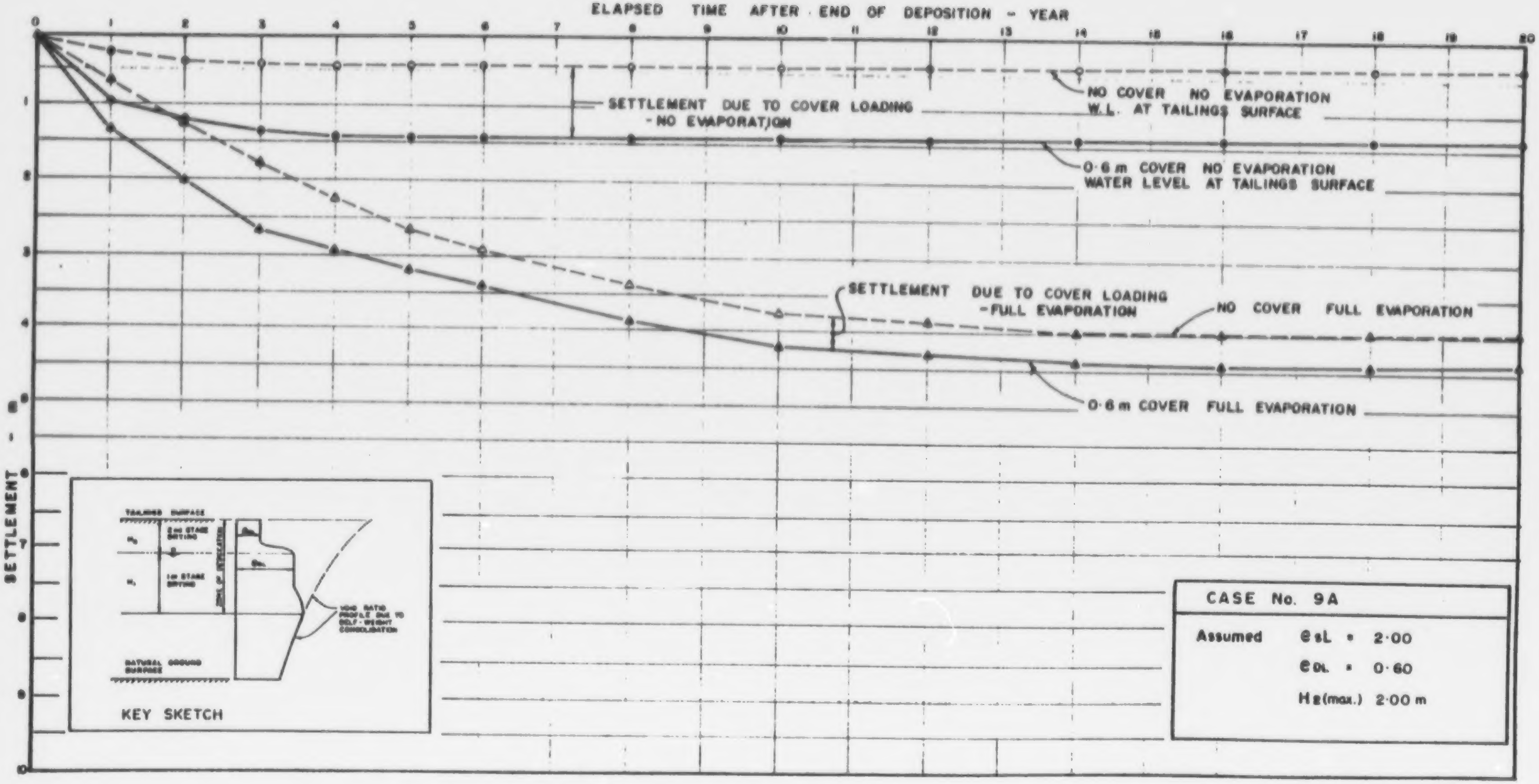


FIGURE 4-12A

SETTLEMENT OF COVER CONSTRUCTEL IMMEDIATELYAFTER END OF TAILINGS DEPOSITION

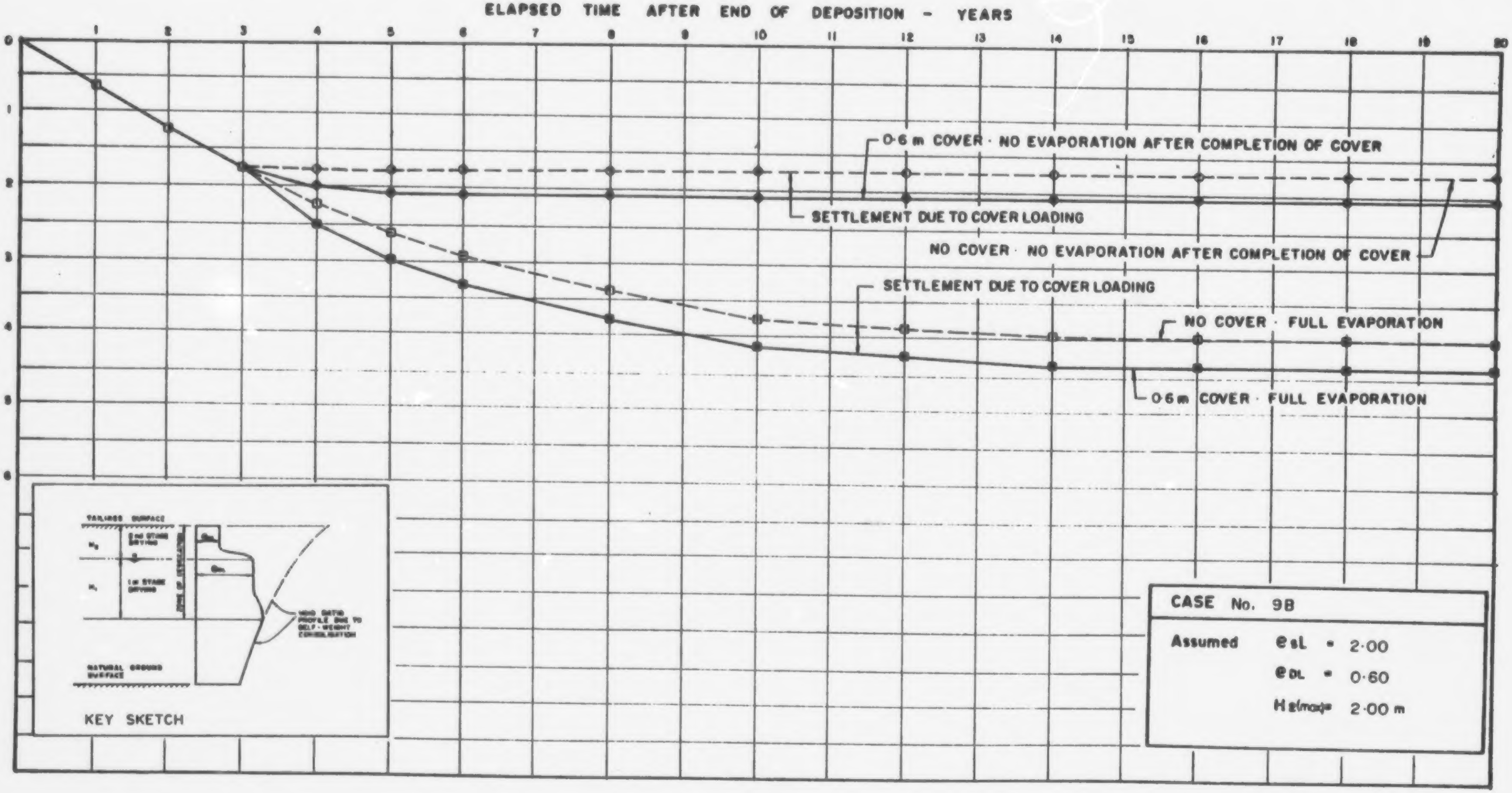


FIGURE 4-12B

SETTLEMENT OF COVER CONSTRUCTED 3 YEARS AFTER END OF TAILINGS DEPOSITION

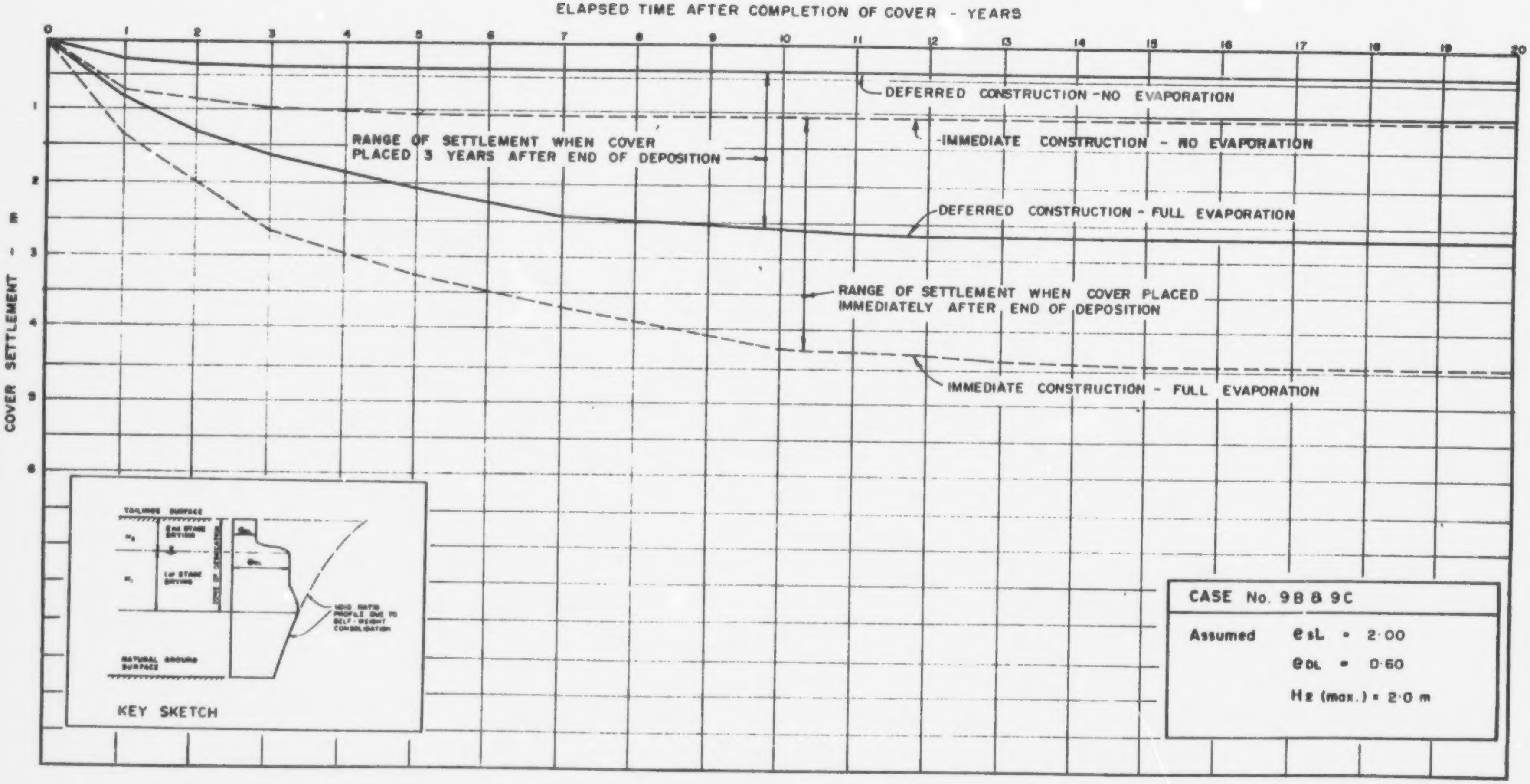


FIGURE 4-12C
REDUCTION OF COVER SETTLEMENTS
BY DEFERRING COVER CONSTRUCTION

5.0 MODELLING OF TYPICAL CASES

Covering of uranium tailings as a decommissioning measure is a relatively recent development in the Canadian scene. No dedicated effort has been made, before the work initiated in this study, in investigating the large strain behaviour of, and desiccation effects on, post-decommissioning settlements of uranium tailings in Canada. Consequently, a thorough search of public archives, accessible industry records and general published technical literature failed to produce sufficient information for definitive modelling of typical cases representative of the high, medium and low grade tailings required in this part of the study. Best use has been made of what information is available and together with some assumed parameters, preliminary models of a somewhat hypothetical nature were developed mainly to highlight the potential magnitudes of the problem and areas of concern and uncertainty which require further investigation in connection with any specific project.

5.1 Medium Grade Neutral Tailings - The Rabbit Lake Model

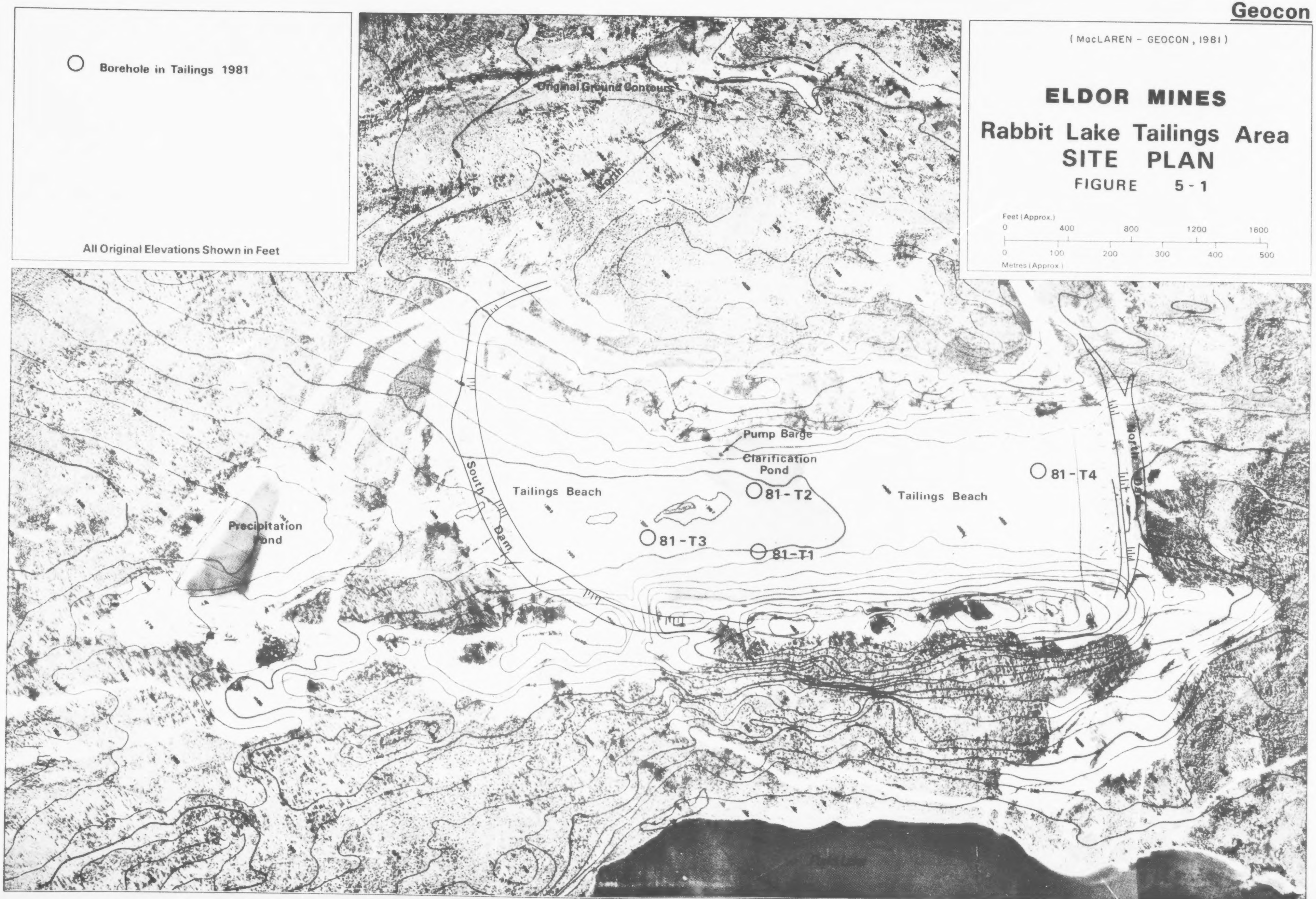
5.1.1 Background Information

The Rabbit Lake Mine is located about 750 km north of Saskatoon, Saskatchewan. It is in an area of sporadic permafrost with short, warm summers and long, cold winters with about 5500 degree days below freezing. Annual average precipitation is about 546 mm with 403 mm in the form of rainfall. Detailed site specific meteorological data recorded at the Collins Bay weather station is available since 1973. No evaporation records are kept at this station. From the Hydrological Atlas of Canada, an average annual evaporation rate of 514 mm was estimated. The nearest meteorological station where monthly Class A pan evaporation records are available is at Cree Lake. For present purposes therefore, the annual evaporation at the site is broken down into monthly values by pro-rating to the Cree Lake statistics. This results in the following rates used for input into the analyses:

Month	Evaporation - mm
January-April	0
May	118
June	119
July	128
August	98
September	51
October-December	0

The tailings produced by an acid leach process at an average rate of about 2000 tonnes per day, is impounded between two dams across the end of a low, broad north-south trending saddle valley as shown in Figure 5-1 on the following page. The North Dam was constructed of compacted low permeability till fill with a system of internal drains constructed of sand and gravel and rock fill to enhance structural stability. The South Dam was initially constructed as a zoned till fill and cyclone tailings sand fill dam with a system of sand and gravel internal drains. However, the use of the tailings sand fill was terminated in 1980 and the balance of the construction carried out with compacted till fill only.

The original valley floor was covered by a variable thickness of peat underlain by a thin overburden consisting of silty to sandy till with an upper zone of bouldery ablation till. The ridges flanking the valley consist of gneissic bedrock with a thin mantle of earth cover. A more detailed description of the site conditions is given in Gulf Minerals, 1980b.


Original Ground Contours
North
Pump Barge
Clarification Pond
81 - T4
81 - T2
81 - T3
81 - T1
Tailings Beach
Tailings Beach
South Dam
North Dam
Precipitation Pond
Feet (Approx.)
0
400
800
1200
1600
0
100
200
300
400
500
Metres (Approx.)

(MacLAREN - GEOCON, 1981)

ELDOR MINES

Rabbit Lake Tailings Area
SITE PLAN
FIGURE 5 - 1

○ Borehole in Tailings 1981

All Original Elevations Shown in Feet

Tailings deposition in the area began in 1975 with an open-ended discharge point located on natural high ground near the west end of the North Dam. Intermittent operation of a mobile cyclone on the South Dam produced sand tailings for raising the South Dam. Underflow from the cyclone, consisting mainly of slimes, was discharged immediately upstream of the South Dam. A pump barge located about mid-way between the North Dam and the South Dam as shown in Figure 5-1 pumps the clarified water via a pipeline to a precipitation pond located to the south of the South Dam. This manner of operation was maintained until some time in 1980 when the mobile cyclone was phased out and the balance of the South Dam constructed using compacted till fill only. From about this time also, point discharge of total tailings from the South Dam and the ridge on the east side was also carried out.

The manner of deposition described above results in a clarification pond (and consequently concentration of slimes) occupying essentially the entire southern half of the area with free water against the South Dam during the initial period up to 1980. Since then, discharge of tailings from the South Dam displaced the south limit of the clarification pond northwards and formed a beach of sandy tailings against the South Dam. The resulting general configuration remains essentially unchanged to date and can be seen in the photo-mosaic of the tailings area shown in Figure 5-1.

A program of field sampling of the tailings from the Rabbit Lake Tailings Area was carried out in the winter of 1981. Four boreholes were put down from the frozen surface of the tailings area at locations indicated in Figure 5-1. Detailed information from that program is reported in Gulf Minerals (1981b).

Frequent layers of frozen tailings were encountered in Borehole 81-T4 put down in the north tailings beach area between the Clarification Pond and the North Dam. Within the clarification pond area and in the south beach area immediately bordering the clarification pond, the tailings was found to be essentially free from frozen layers. This pattern of distribution of frozen tailings is consistent with that anticipated from the history of deposition described earlier.

The two boreholes 81-T1 and 81-T2 located within the Clarification Pond area did not produce reliable moisture content profiles due to difficulties experienced in recovering samples from the upper very loose zone of freshly settled tailings. Borehole 81-T3 however, yielded good samples and the moisture content and void ratio profiles obtained from them are shown in Figure 5-2 on the following page. Excess porewater pressures within the tailings obtained from piezometers during the field program are also shown in the Figure. The results of two oedometer tests carried out on Shelby tube samples of silty tailings (slimes) are shown in Figures 5-3 and 5-4 on the following pages. Other geotechnical properties of the tailings (Gulf Minerals 1981b) are summarized below.

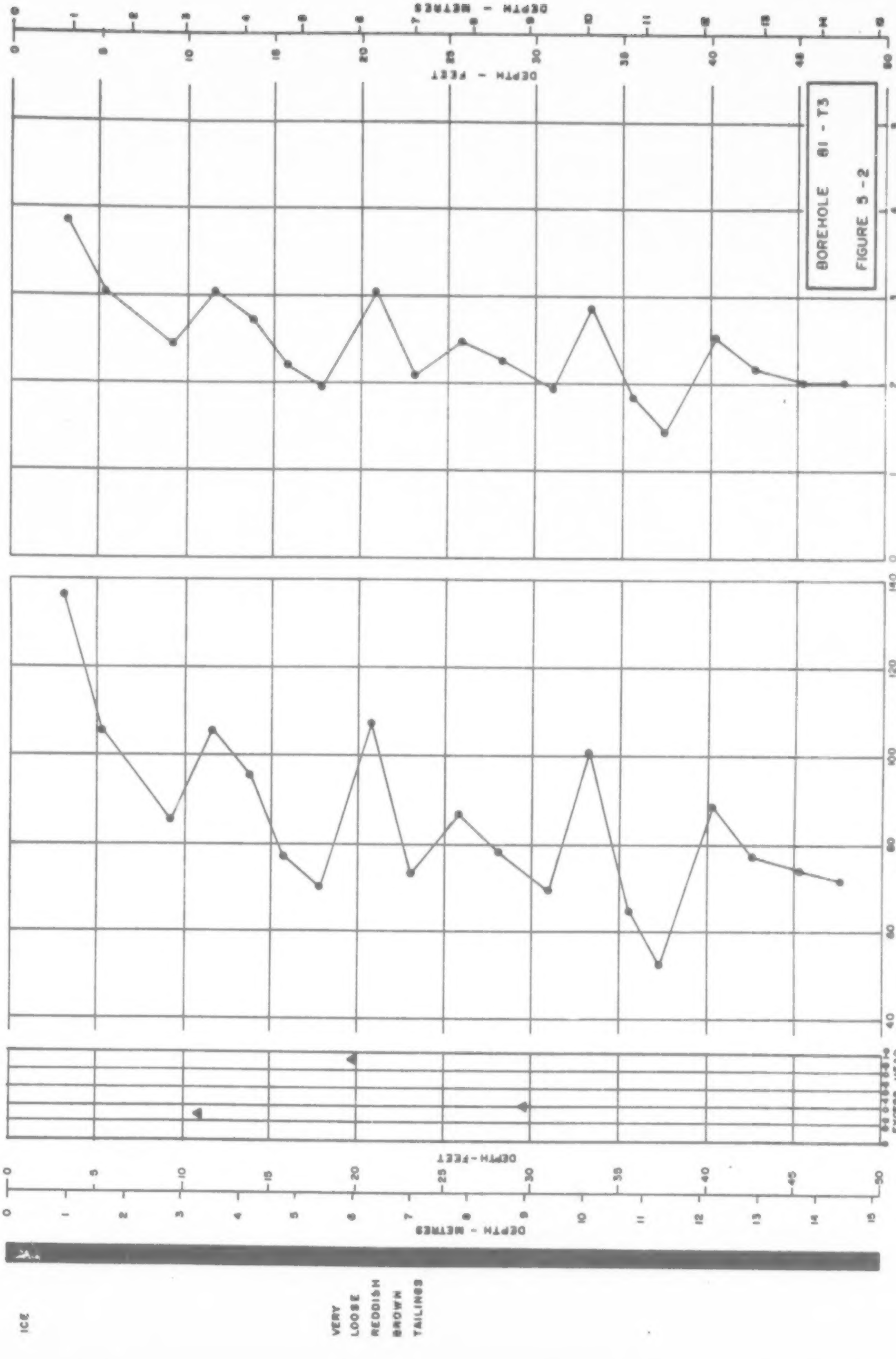
Grainsize Characteristics

Sandy Tailings	
Percentage - No. 200	47 to 55
Percentage - 2 μm	less than 2
D_{50}	0.069 to 0.013
Silty Tailings	
Percentage - No. 200	82 to 99
Percentage - 2 μm	3 to 10
D_{50}	0.069 to 0.013

Atterberg Limits

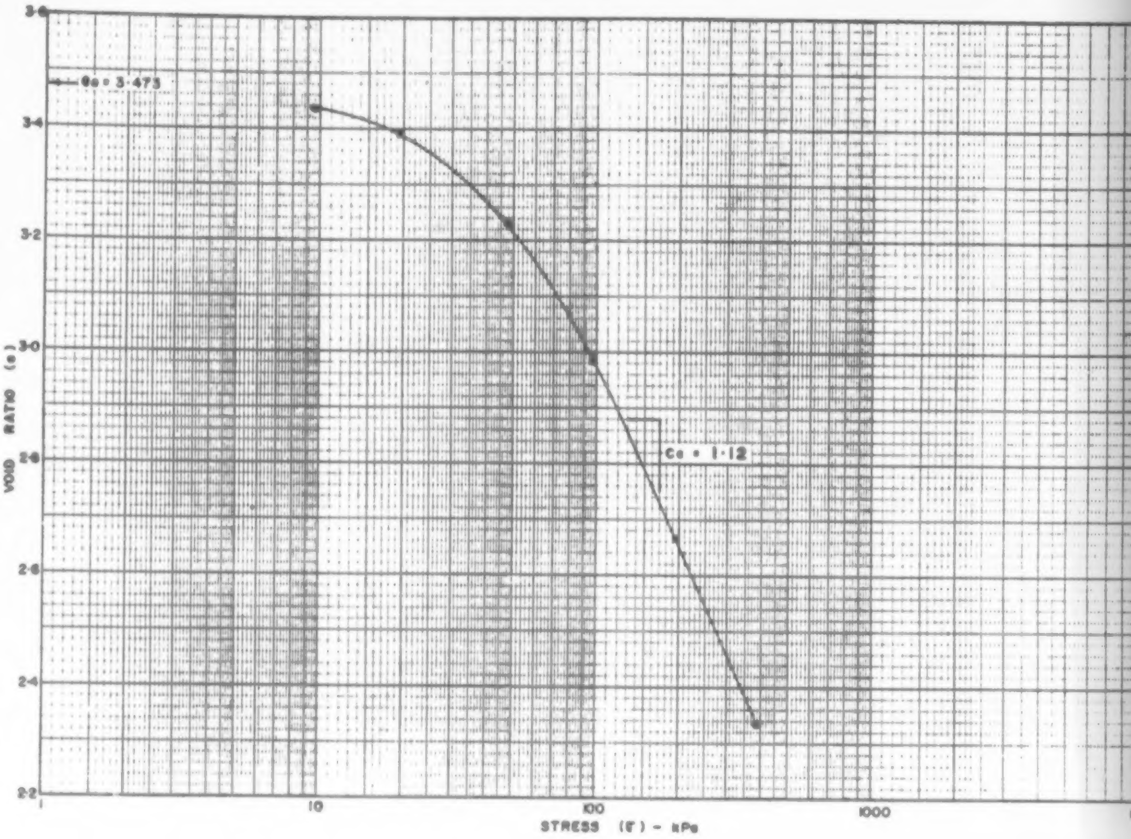
Silty Tailings	
Liquid limit	54 to 87
Plastic limit	41 to 66

Specific Gravity of Solids	2.8 to 3.0



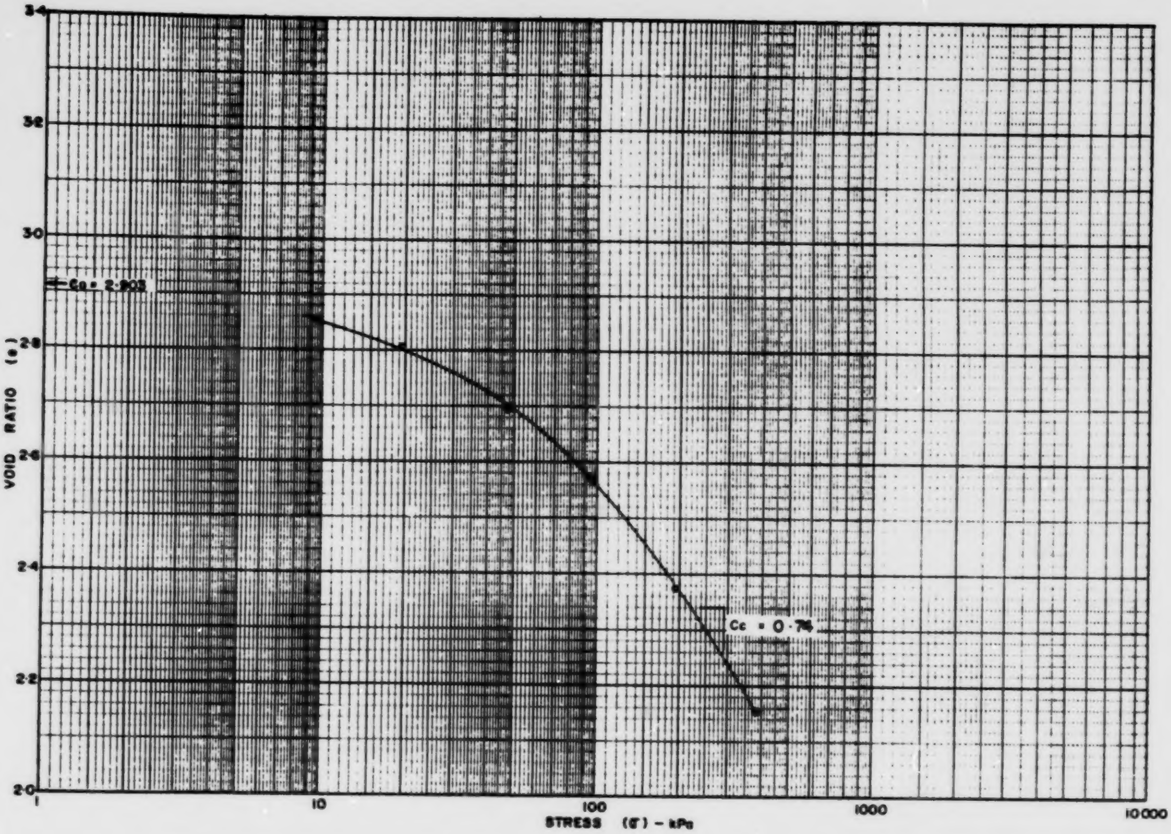
BOREHOLE 81 - T3

FIGURE 5-2



σ' (kPa)	e	c_v (m /day)	m_v (kPa^{-1})	k (m / sec)
0·0	3·473			
		0·134	0·00089	$1{\cdot}348 \times 10^{-8}$
9·58	3·435			
		0·132	0·00104	$1{\cdot}552 \times 10^{-8}$
19·15	3·391			
		0·036	0·00129	$5{\cdot}276 \times 10^{-9}$
47·88	3·228			
		0·110	0·00122	$1{\cdot}522 \times 10^{-8}$
95·76	2·981			
		0·093	0·00083	$8{\cdot}744 \times 10^{-9}$
191·52	2·665			
		0·076	0·00050	$4{\cdot}323 \times 10^{-9}$
383·04	2·313			

FIG. 5-3 OEDOMETER TEST RESULT –
BOREHOLE 81 – T3 SAMPLE NO. 14



σ' (kPa)	e	c_v (m /day)	m_v (kPa^{-1})	k (m / sec)
0·0	2·903			
		0·110	0·00128	$1{\cdot}602 \times 10^{-8}$
9·58	2·855			
		0·130	0·00125	$1{\cdot}839 \times 10^{-8}$
19·15	2·809			
		0·111	0·00104	$1{\cdot}312 \times 10^{-8}$
47·88	2·695			
		0·185	0·00068	$1{\cdot}435 \times 10^{-8}$
95·76	2·574			
		0·103	0·00058	$6{\cdot}759 \times 10^{-9}$
191·52	2·376			
		0·127	0·00034	$4{\cdot}968 \times 10^{-9}$
383·04	2·153			

FIG. 5-4 OEDOMETER TEST RESULT –
BOREHOLE 81 - T1 SAMPLE NO. 9

Void ratio - effective stress and void ratio - permeability relationships from field and laboratory test data have been described earlier and shown in Figures 4-1 and 4-2, respectively.

The grainsize characteristics indicated above are inconsistent with the compressibility characteristics obtained in the oedometer tests and field void ratio profiles. This is believed to be due to the presence of needle-like gypsum particles formed during the neutralization process by lime treatment prior to disposal in the tailings impoundment area.

5.1.2 Idealized Model

For the purpose of this study, an idealized model with simplified geometry and assumed material zoning is used, fashioned more-or-less after the overall dimensions and history of deposition of the Rabbit Lake tailings area. A typical longitudinal section and a transverse section through the clarification pond area, are shown in Figure 5-5.

The north end of the area is assumed to consist predominantly of frozen tailings. Whatever unfrozen zones that may be present are assumed to be sandy tailings which is fully consolidated at the end of deposition. Any post-decommissioning settlements within this area can therefore be assumed to be negligible compared to those which are anticipated to occur within the clarification pond area. This is provided of course that the frozen tailings does not thaw out in the long term. Except in the event of a change towards warmer climates, this is a reasonable assumption because the heat input during active deposition to the area is considerably greater than after decommissioning of the facility.

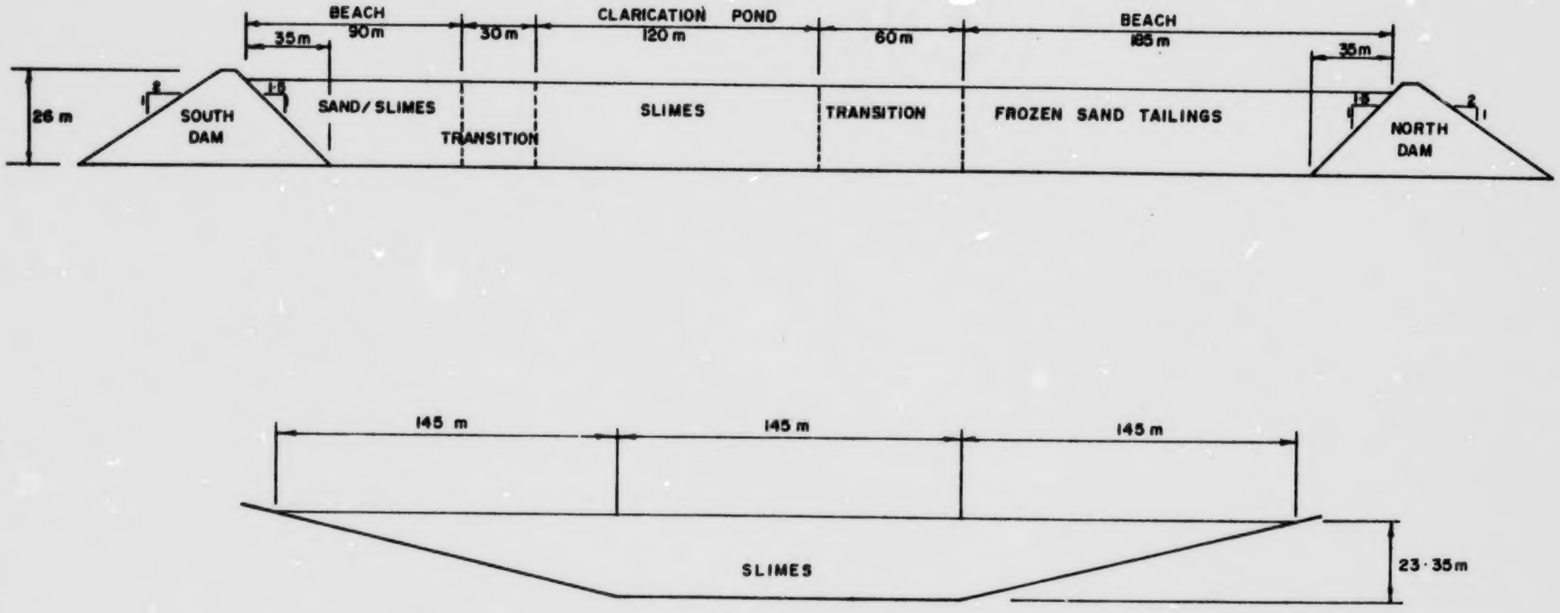

BEACH
90m
35m
30m
CLARICATION POND
120m
60m
BEACH
185m
35m
26m
2
1
1.5
1
SOUTH DAM
SAND/SLIMES
TRANSITION
SLIMES
TRANSITION
FROZEN SAND TAILINGS
1.5
1
2
1
NORTH DAM
145m
145m
145m
SLIMES
23·35m

FIGURE 5 - 5

IDEALIZED MODEL OF TAILINGS AREA
RABBIT LAKE

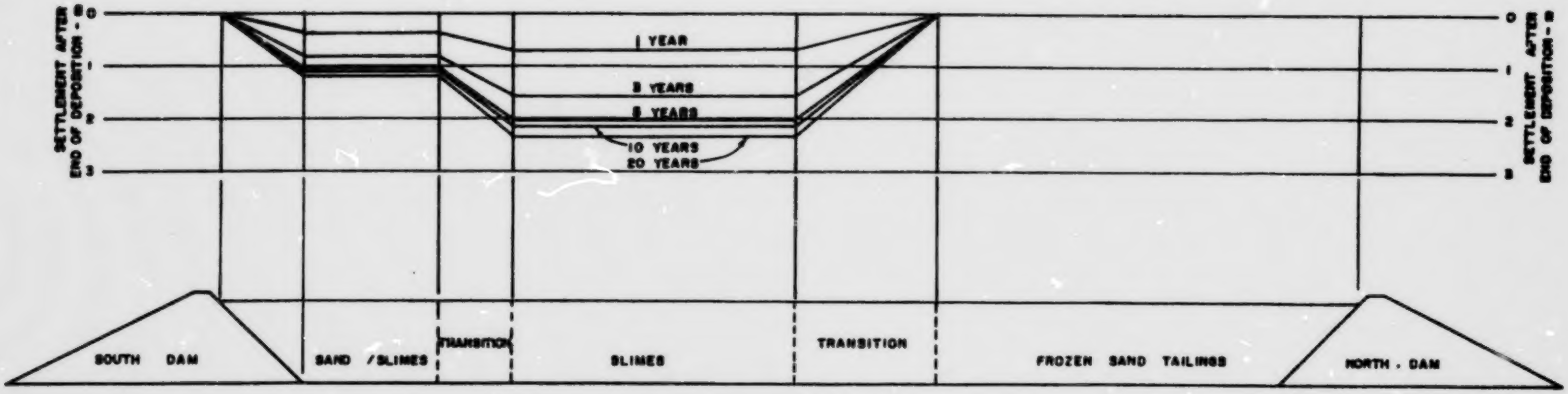
The central area consisting of silty tailings (slimes) deposited within the clarification pond is reprsented by the one-dimensional consolidation model described in detail in Section 4.0. As discussed therein, the sensitivity of long term settlements to desiccation parameters makes it impossible to obtain a definitive solution of the consolidation problem. The maximum depth of desiccation, in particular, has the greatest impact on long term settlements. In general terms, however, it is expected that surrounding hydrogeological conditions will prevent lowering of the water level within the tailings to great depths. Special attention could also be given in the design of the cover to minimize long term evaporation effects, as discussed later. Thus, the case of

e_{sL} = 3.59
e_{DL} = 1.71
H_2 (max) = 2.0 m

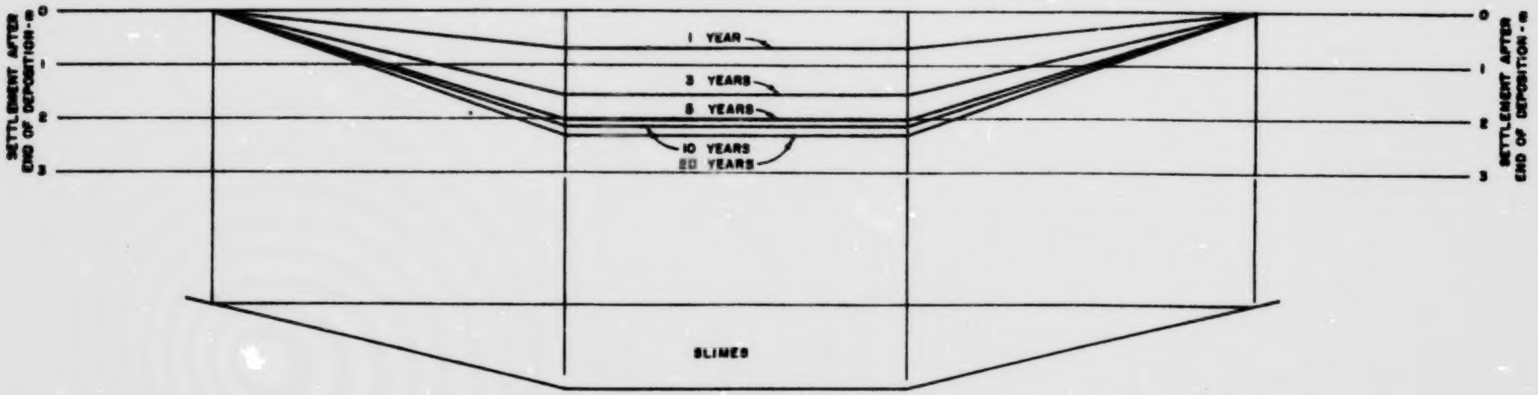
is selected for discussion herein. The self-weight consolidation settlements and rate of lowering of the water level due to desiccation in this case has been shown in Figure 4-7C and the settlements under a simulated cover of 0.6 m thickness, in Figures 4-11. Placement of the cover immediately following completion of tailings deposition is anticipated to be difficult and hazardous if not impossible due to the very soft saturated condition of the tailings at this time. The effects of desiccation shown in Figure 4-7C clearly demonstrates the advantage of a deferred construction of the cover. For instance, after a period of three years, the water level will have receded to more than 1 m below the surface of the tailings, forming a desiccated crust which should have sufficient strength for supporting construction equipment and fill placed in construction of the cover. In addition, shrinkage due to desiccation of the tailings above the water level and consolidation of the saturated tailings due to the surcharging effect from lowering of the water level over this period will have occurred, thereby significantly reducing the post-construction settlements of the cover. To illustrate this effect the cover settlements for the case of a 3-year deferment of construction are shown in Figure 4-11B and a comparison with the hypothectical case of immediate construction following tailings deposition, shown in Figure 4-11C. The area adjacent to the South Dam is anticipated to contain an upper layer of sandy tailings and a lower layer of predominantly silty tailings (slimes). The computer program PCDDDF does not have the capability to analyse multi-layered consolidating materials, nor does any other currently available computer programs based on large strain consolidaiton theories. For present purposes, it is assumed that the settlements in this area will be 50 percent of the corresponding settlements in the clarification pond area.

To complete the picture, settlements within the transition zones are assumed to vary linearly between the beach areas and the clarification pond area.

Based on the above approach, settlement profiles at various time intervals are shown in Figures 5-6 through 5-10 shown on the following pages. Figure 5-10 shows the settlement profiles of the tailings surface without cover over the long term under the full effects of evaporation desiccation. As indicated earlier, placement of a cover over the tailings surface will impede the rate of evaporation and decrease the depth of water level lowering within the tailings with accompanied decreases in long term settlements. The effectiveness of the cover in moderating evaporation effects will depend on its design, which of course makes it a very important consideration in planning decommissioning measures. The significance of this effect can be appreciated by comparing Figure 5-7 to Figure 5-8 and comparing Figure 5-9 to Figure 5-10. The effectiveness of a deferred cover construction is demonstrated by comparing Figure 5-7 to Figure 5-9 and comparing Figure 5-8 to Figure 5-10.

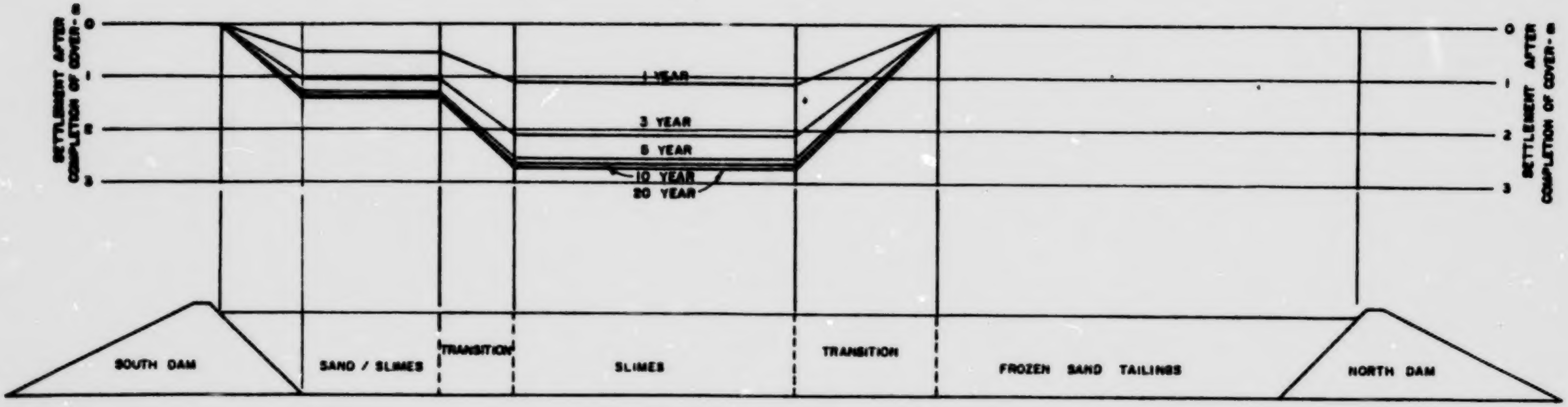


LONGITUDINAL SECTION

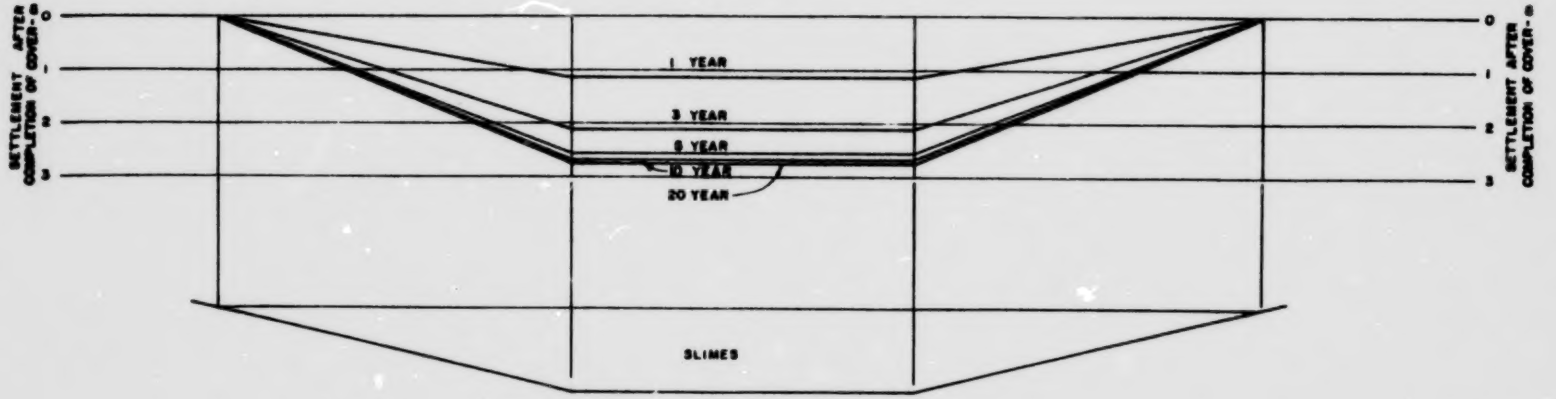


TRANSVERSE SECTION

FIGURE 5 - 6
SETTLEMENT PROFILES
FULL EVAPORATION NO COVER

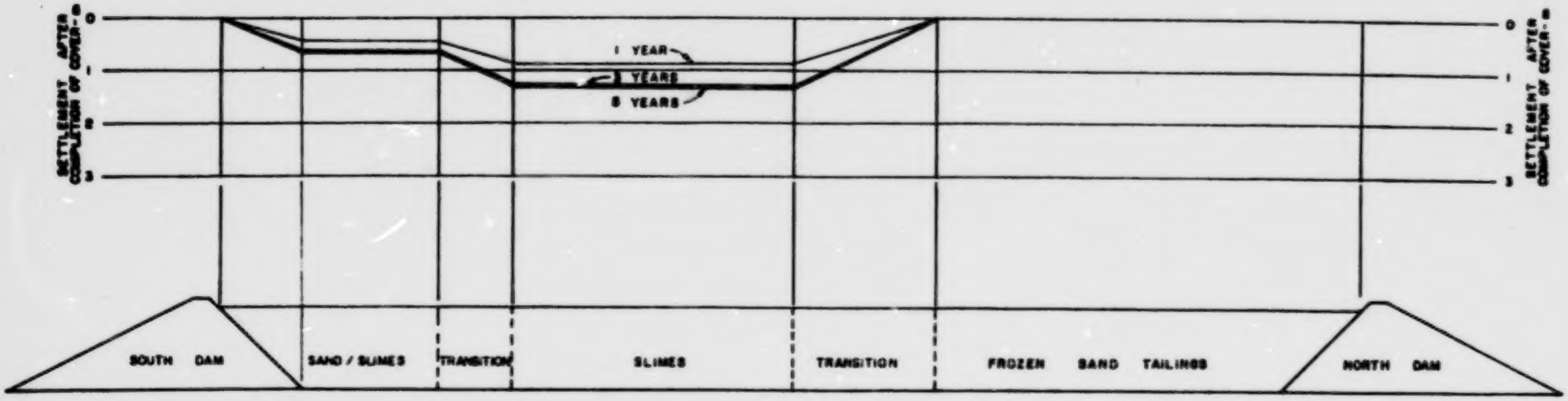


LONGITUDINAL SECTION

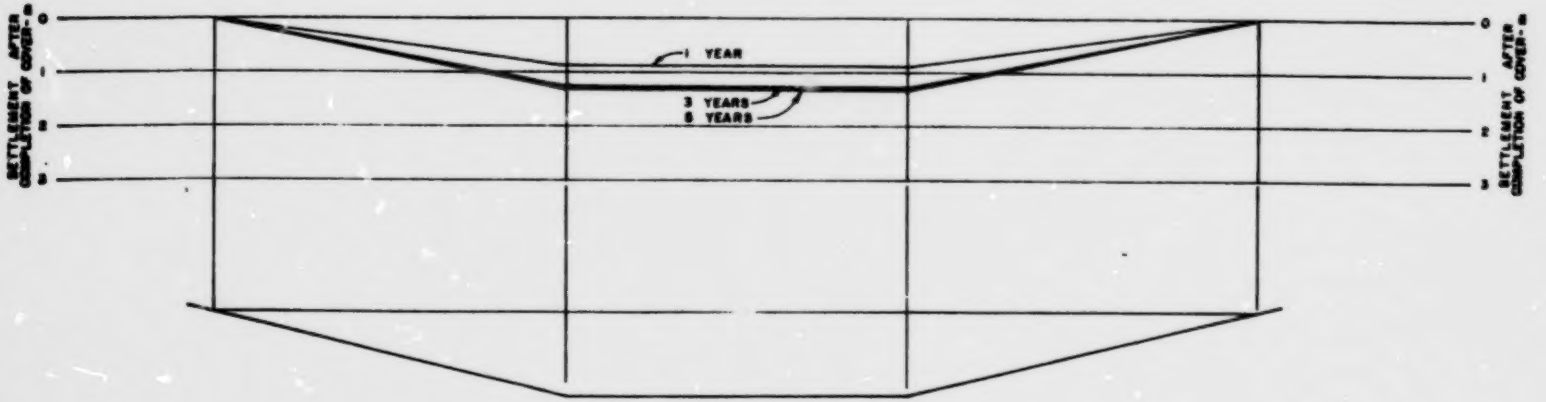


TRANSVERSE SECTION

FIGURE 5-7
SETTLEMENT PROFILES
FULL EVAPORATION
COVER CONSTRUCTED IMMEDIATELY
FOLLOWING TAILINGS DEPOSITION

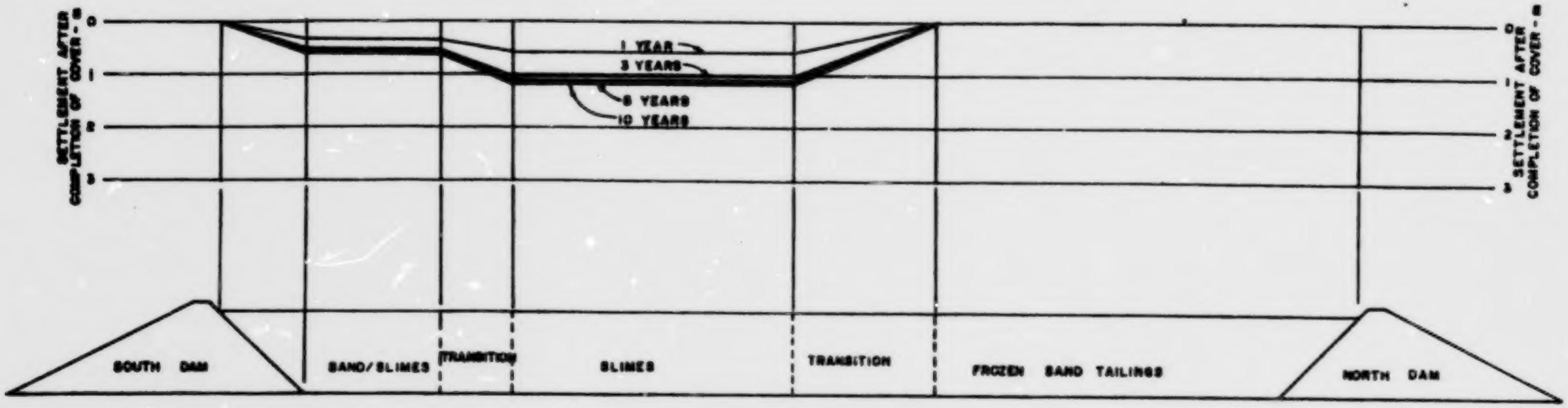


LONGITUDINAL SECTION

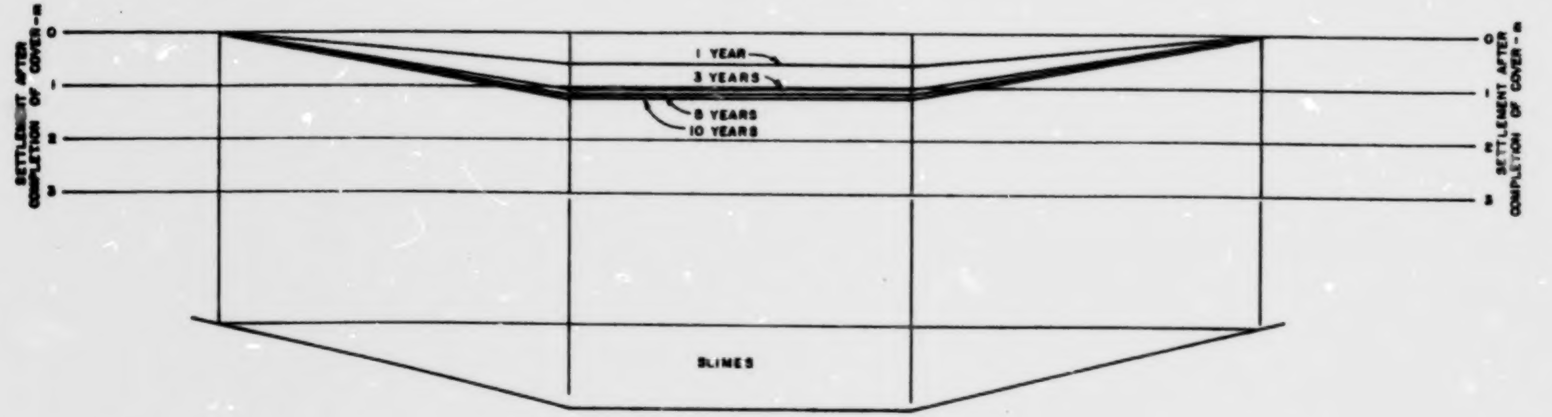


TRANSVERSE SECTION

FIGURE 5 - 8
SETTLEMENT PROFILES
NO EVAPORATION
COVER CONSTRUCTED IMMEDIATELY
FOLLOWING TAILINGS DEPOSITION

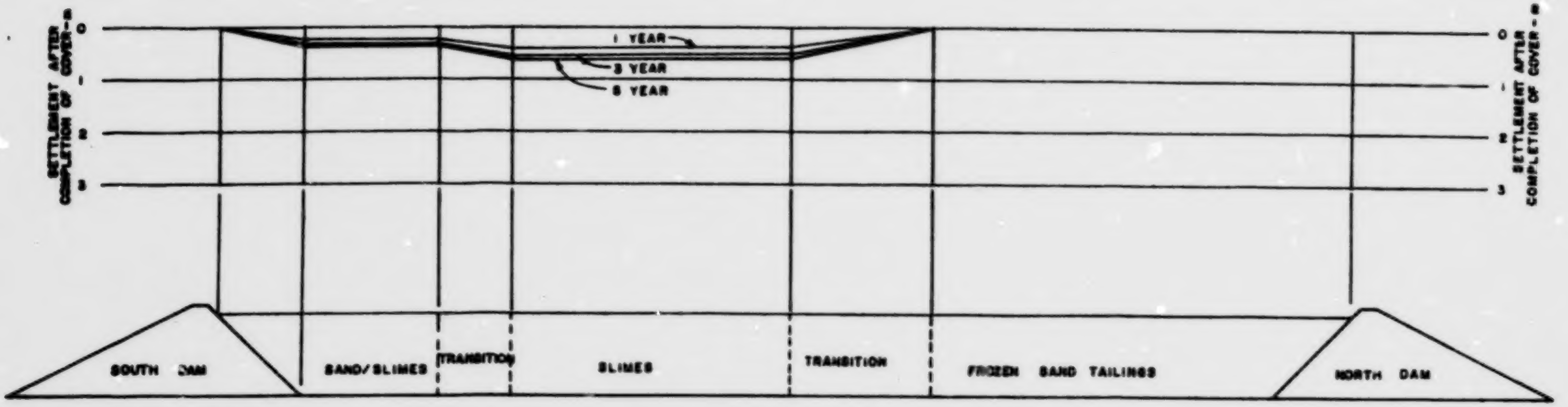


LONGITUDINAL SECTION

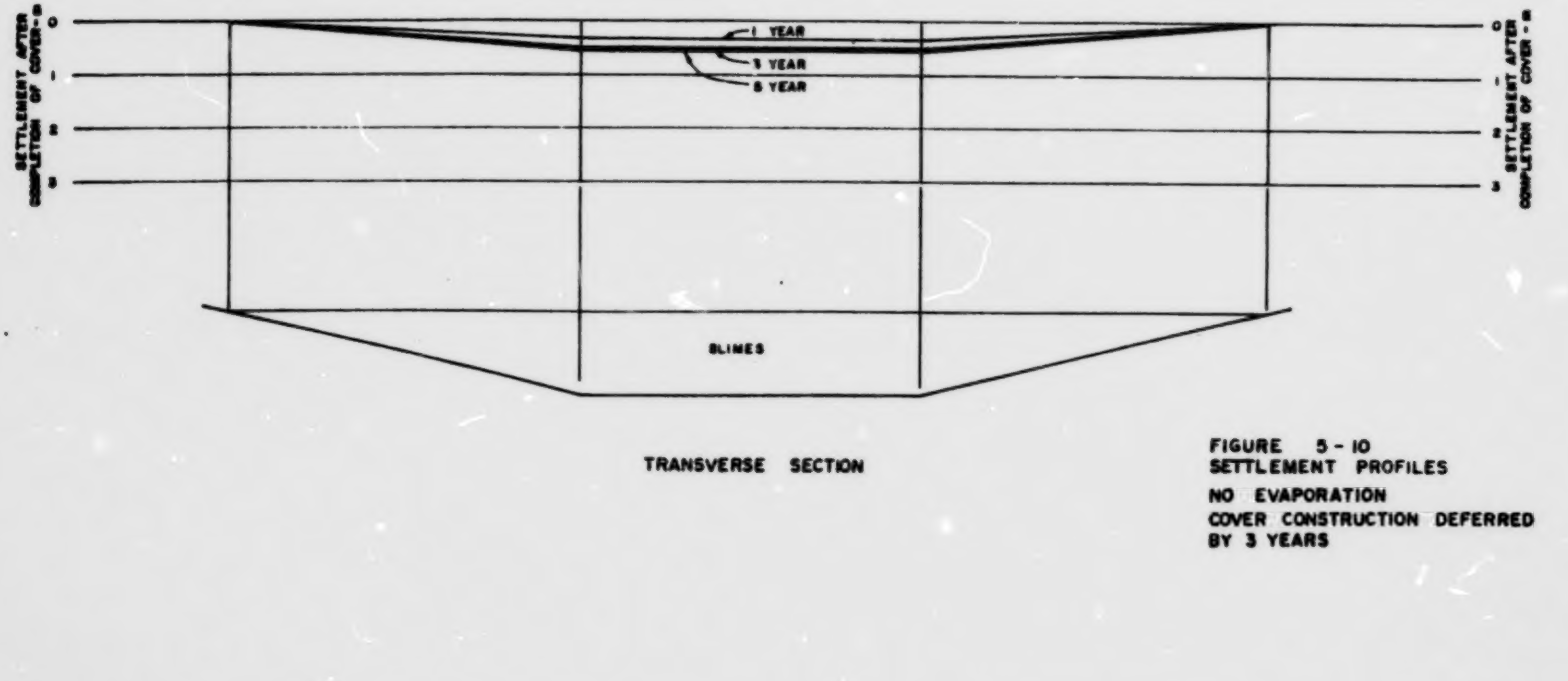


TRANSVERSE SECTION

FIGURE 5 - 9
SETTLEMENT PROFILES
FULL EVAPORATION
COVER CONSTRUCTION DEFERRED
BY 3 YEARS



LONGITUDINAL SECTION



TRANSVERSE SECTION

FIGURE 5 - 10
SETTLEMENT PROFILES
NO EVAPORATION
COVER CONSTRUCTION DEFERRED
BY 3 YEARS

5.2 Low Grade Pyritic Tailings - Elliot Lake Model

5.2.1 Background Information

The bulk of uranium mine tailings in Canada, about 81 percent, is located in the Elliot Lake area of Ontario where mining operations have produced about 100 million tonnes of tailings since operation began in 1954 (Dance, 1985). It is projected that when mining in this area ends, there will be more than one billion tonnes of tailings produced (Blair et al 1980).

The Elliot Lake area is located in northwestern Ontario between Sudbury and Sault Ste. Marie as shown in Figure 5-11. A total of 12 mines with tailings deposited in 9 impoundments have been in operation in the Elliot Lake area and their locations are shown in Figure 5-12. Of these only the Quirk Mine operated by Rio Algom Limited and the Denison Mine operated by Denison Mines Limited have been in production up to the present time.

The ore deposits in the Elliot Lake area belong to the detrital (reef) type. These deposits are large, relatively low-grade and occur in a quartz pebble conglomerate. The principal accessory mineral in the ore is pyrite (iron sulphide) and the uranium mineralization occurs in three uranium bearing minerals: brannerite, uraninite and monazite (Dance 1985).

The grind of the ore averages 50%, minus 200 mesh sizes and an acid-leach and ion-exchange process is used. The residual tailings and the process effluent water are then neutralized with lime treatment before being discharged into the tailings areas.

Information on the geotechnical properties of the Elliot Lake tailings pertinent to the present study were extracted from Dance (1985) Matyas et al (1984) and Golder Associates (1977). Most of this information was obtained from tests carried out on tailings produced from the Denison Mine.

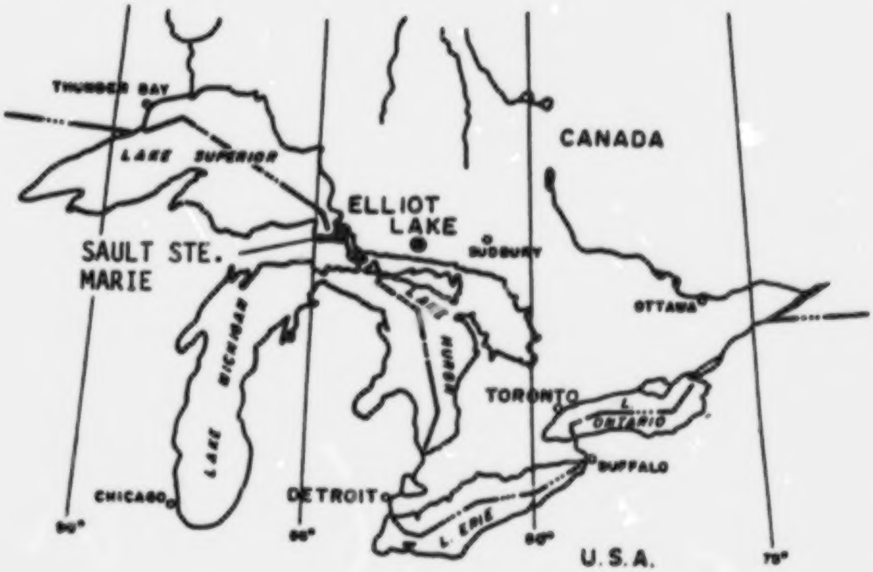

THUNDER BAY
LAKE SUPERIOR
CANADA
ELLIOT LAKE
SAULT STE. MARIE
SUDBURY
OTTAWA
LAKE MICHIGAN
LAKE HURON
TORONTO
L. ONTARIO
BUFFALO
CHICAGO
DETROIT
L. ERIE
90°
85°
80°
U.S.A.
75°

FIGURE 5.11 - LOCATION OF ELLIOT LAKE AREA

(After Canmet - NUTP, 1985)

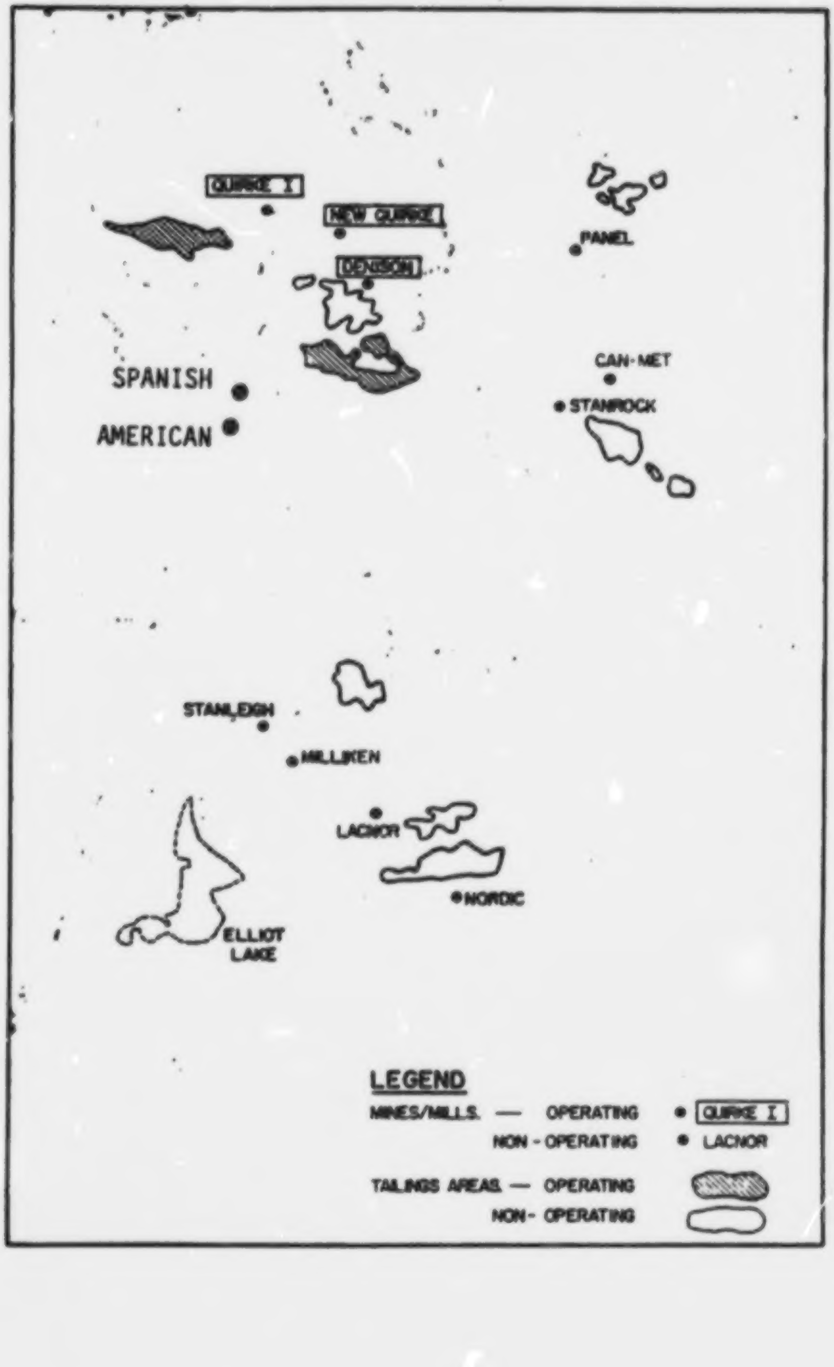

QUIRKE I
NEW QUIRKE
DENISON
PANEL
SPANISH
AMERICAN
CAN-MET
STANROCK
STANLEIGH
MILLIKEN
LACNOR
NORDIC
ELLIOT
LAKE
LEGEND
MINES/MILLS — OPERATING
QUIRKE I
NON-OPERATING
LACNOR
TAILINGS AREAS — OPERATING
NON-OPERATING

FIGURE 5.12 - ELLIOT LAKE AREA

(After Gorber et. al., 1978)

In general, the tailings are highly angular with the presence of needle-shaped gypsum crystals precipitated from the lime-treatment process. Typical grainsize distribution curves given in Matyas, et al (1984), are reproduced in Figure 5-13, on the following page. Average Liquid Limit of 33, Plastic Limit of 31 and Plasticity Index of 2 reported by Dance (1985) indicate that the tailings are non-plastic.

The average specific gravity of the sandy tailings is 2.73. The specific gravity of the silty tailings is in the range between 2.55 and 2.80 with an average of 2.67 (Dance, 1985).

Results of oedometer tests given in Matyas et al (1984) and Golder (1977) are shown in Figure 5-14.

Field and laboratory permeability test results as reported by Matyas et al (1984) are plotted against the corresponding void ratios as shown in Figure 5-15.

For the purpose of this study site conditions and history of tailings deposition at the Long Lake project is used for general guidance in setting up the idealized model mainly because of the availability of moisture content and density profiles from boreholes put down through the tailings (CANMET-NUTP, 1985) which are reproduced in Figure 5-16. As discussed earlier, such information provides the best estimate of the consolidation behaviour en-massse which enables better estimates of long term consolidation settlements to be made in modelling analyses.

The Long Lake tailings area is characterized by the presence of bedrock ridges at the north and south sides. The bedrock ridges consist predominantly of relatively impervious metamorphosed conglomerates, siltstones

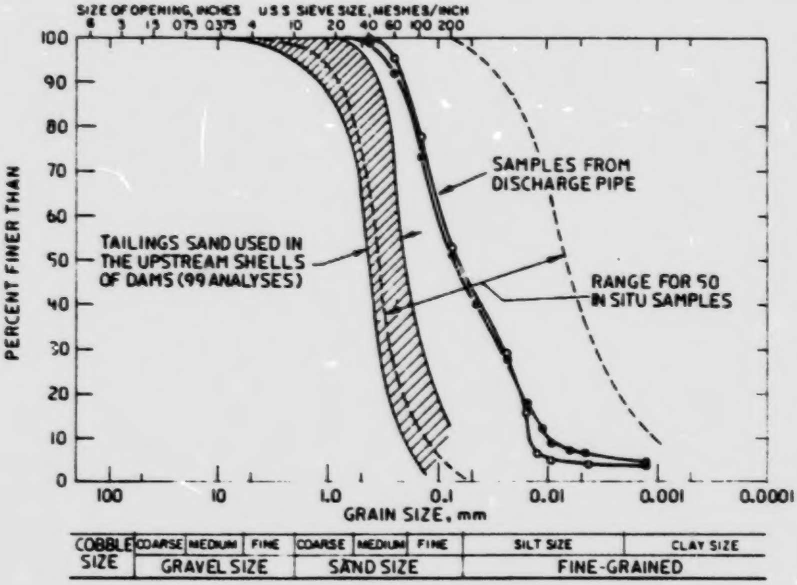


FIGURE 5.13 - GRAIN SIZE DISTRIBUTION
ELLIOT LAKE URANIUM TAILINGS

(After Matyas et. al. 1984)

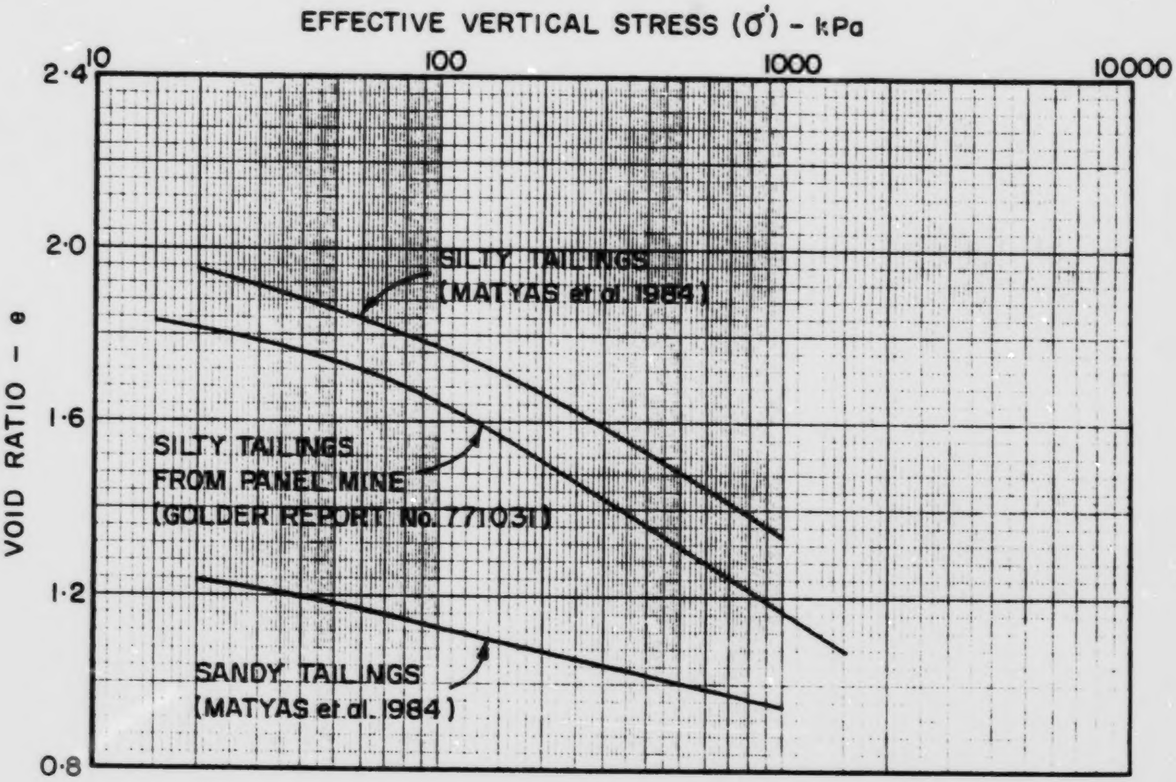


FIGURE 5.14 - CONSOLIDATION CHARACTERISTICS OF THE ELLIOT LAKE TAILINGS

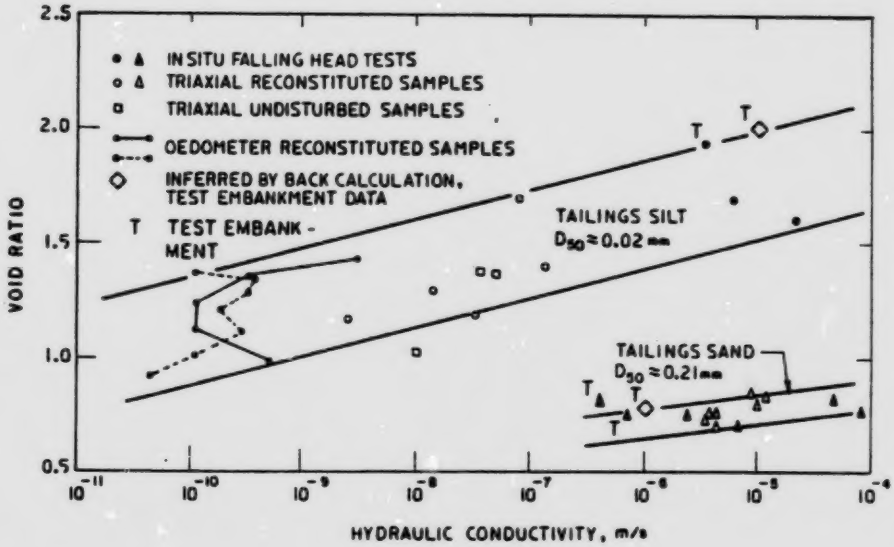


FIGURE 5.15 e-k RELATIONSHIPS FROM MATYAS ET AL 1984
ELLIOT LAKE

FIGURE 5.15

(After Matyas et. al. 1984)

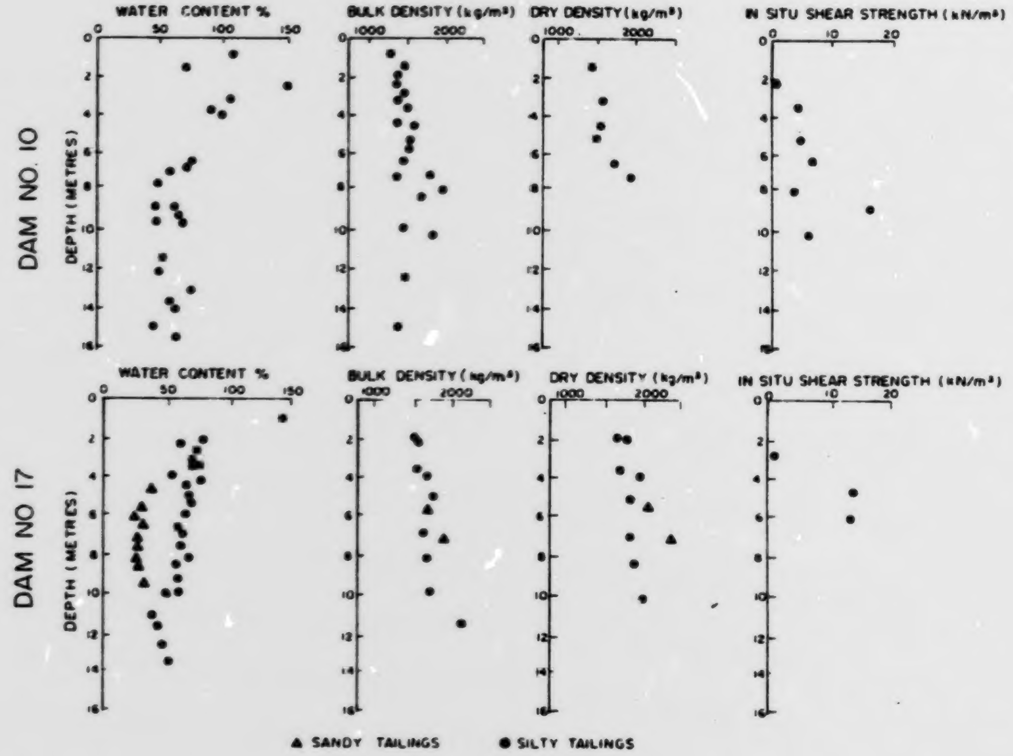


FIGURE 5.16 WATER CONTENT, DENSITY AND IN SITU SHEAR STRENGTH PROFILES - LONG LAKE (AFTER CANMET - NUTP, 1985)

(After Canmet - NUTP, 1985)

and arkoses of the Gowganda Formation. The tailings area is enclosed by Dam No. 10 to the west and Dams Nos. 9 and 17 to the east. Dam No. 10 consists of earthfill with a low permeability synthetic membrane. Dams Nos. 9 and 17 consist of earthfill with low permeability glacial till cores. Figure 5-17 shows the 1977 layout of the Long Lake tailings area and the associated facilities.

The subsurface conditions vary, with exposed sound bedrock near the ridges, a thick layer of very dense and low permeability glacial till underlain by sound bedrock at the east near Dam No. 17 and loose to compact granular subsoil of up to 20 m thick underlain by jointed bedrock at the west near Dam No. 10. Turton (1977) suggested a hydraulic conductivity value of 1×10^{-5} cm/sec for the impervious glacial till and bedrock and 1×10^{-3} cm/sec to 1×10^{-5} cm/sec for the jointed bedrock. For the loose to compact granular subsoil, it is commonly known to have a hydraulic conductivity value in the order of 1×10^{-2} cm/sec. Thin surficial and discontinuous layers of fibrous peat with a hydraulic conductivity value of 5×10^{-5} cm/sec, as suggested by Matyas et al (1984), is also known to be present across the tailings area.

LaRocque and Webber (1978) reported that approximately 20 million tons (dry weight) of tailings were deposited between 1960 and 1976. CANMET (1985) reported that a total of approximately 40 million tonnes covering a plan area of 125 hectares were deposited by 1984.

The in-situ bulk density for the sandy tailings and silty tailings are 19.7 and 16.7 kg/m^3 respectively as reported by Matyas giving an average value of 18.0 kg/m^3. Tailings was reportedly end discharged from the east end of

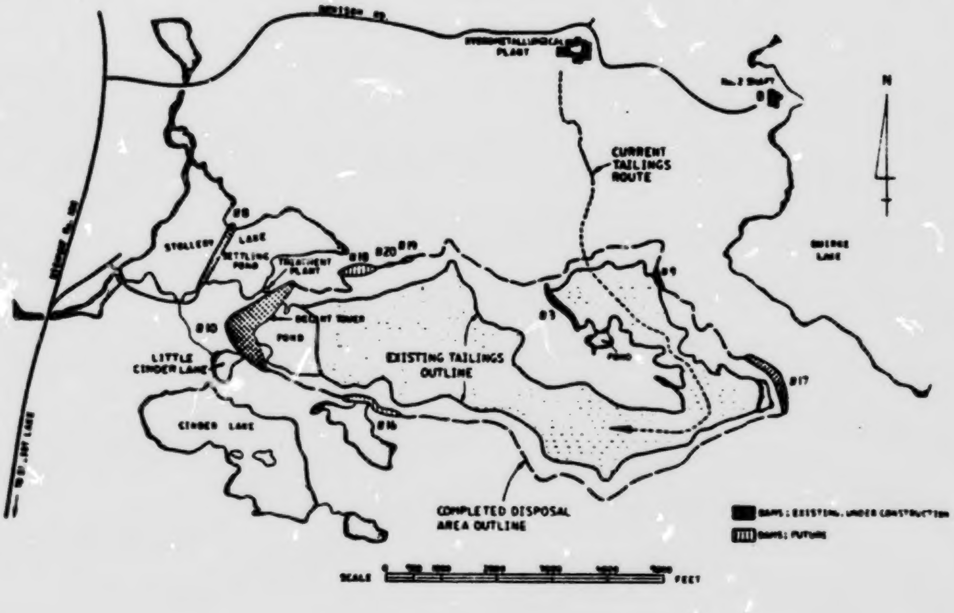

HYDROMETALLURGICAL PLANT
No. 2 SHAFT
N
CURRENT TAILINGS ROUTE
QUIRKE LAKE
D8
STOLLERY LAKE
SETTLING POND
TREATMENT PLANT
D18
D20
D19
D9
D3
D10
DECANT TOWER
POND
EXISTING TAILINGS OUTLINE
POND
LITTLE CINDER LAKE
D17
CINDER LAKE
D16
COMPLETED DISPOSAL AREA OUTLINE
DAMS; EXISTING, UNDER CONSTRUCTION
DAMS; FUTURE
SCALE
0
500
1000
2000
3000
4000
5000
FEET

FIGURE 5.17 - LONG LAKE TAILINGS AREA
1977 LAYOUT
ELLIOT LAKE

(After Turton 1977)

the area as indicated in Figure 5-17. Consequently, the surface of the tailings slopes from east to west at a grade of approximately 0.5 percent and sandy tailings predominate in the east end of the tailings area while silty tailings (slimes) predominate in the west end due to segregation.

5.2.2 Idealized Model

Based on the general information given in 5.2.1 above, an idealized model was developed in the form of a longitudinal section as shown in Figure 5-18.

A one-dimensional consolidation model was used to estimate the settlement of the silty tailings (slimes) in the clarification pond area. The assumed e-σ' and e-k relationships used in the analysis are shown in Figures 5-19 and 5-20, respectively. The rate of accretion was assumed to be 1.623 m per year in Lagrangian height at an initial void ratio of 4.50. Deposition was assumed to be ongoing for a period of 30 years. The accretion and self-weight consolidation model is shown in Figure 5-21. A comparison of the computed height of the tailings at year 19 and the computed void ratio profile to those observed in boreholes put down at about that time is shown in Figure 5-22.

To model the effect of evaporation on long term settlements, the following assumed parametric values were used:

$e_{SL} = 1.65$

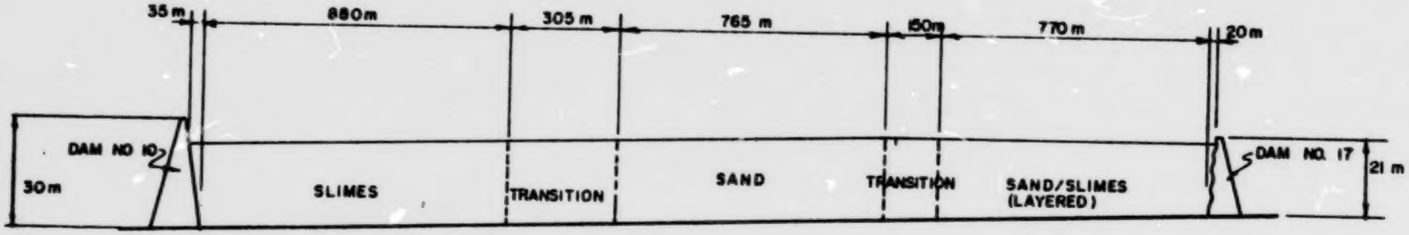
$e_{DL} = 1.00$

$H_2(\max) = 2.00$

$S = 75\%$

$C'_E = 1.00$

$C_D = 1.00$


35 m
880m
305 m
765 m
150m
770 m
20m
30m
DAM NO 10
SLIMES
TRANSITION
SAND
TRANSITION
SAND/SLIMES (LAYERED)
DAM NO. 17
21 m

LONGITUDINAL SECTION

FIGURE 5 - 18

IDEALIZED MODEL OF LONG LAKE TAILINGS AREA - ELLIOT LAKE

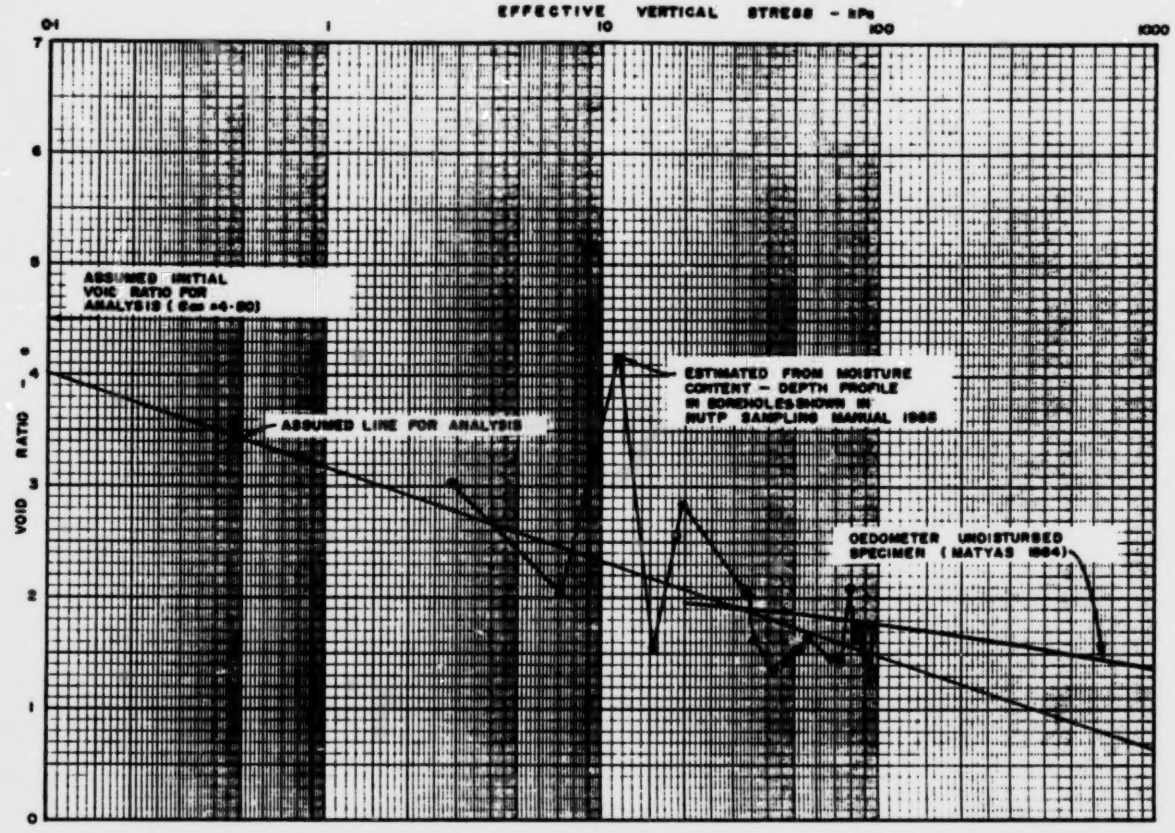


FIGURE 5- 19 $e-\sigma'$ RELATIONSHIPS FROM FIELD AND LABORATORY TEST DATA – ELLIOT LAKE

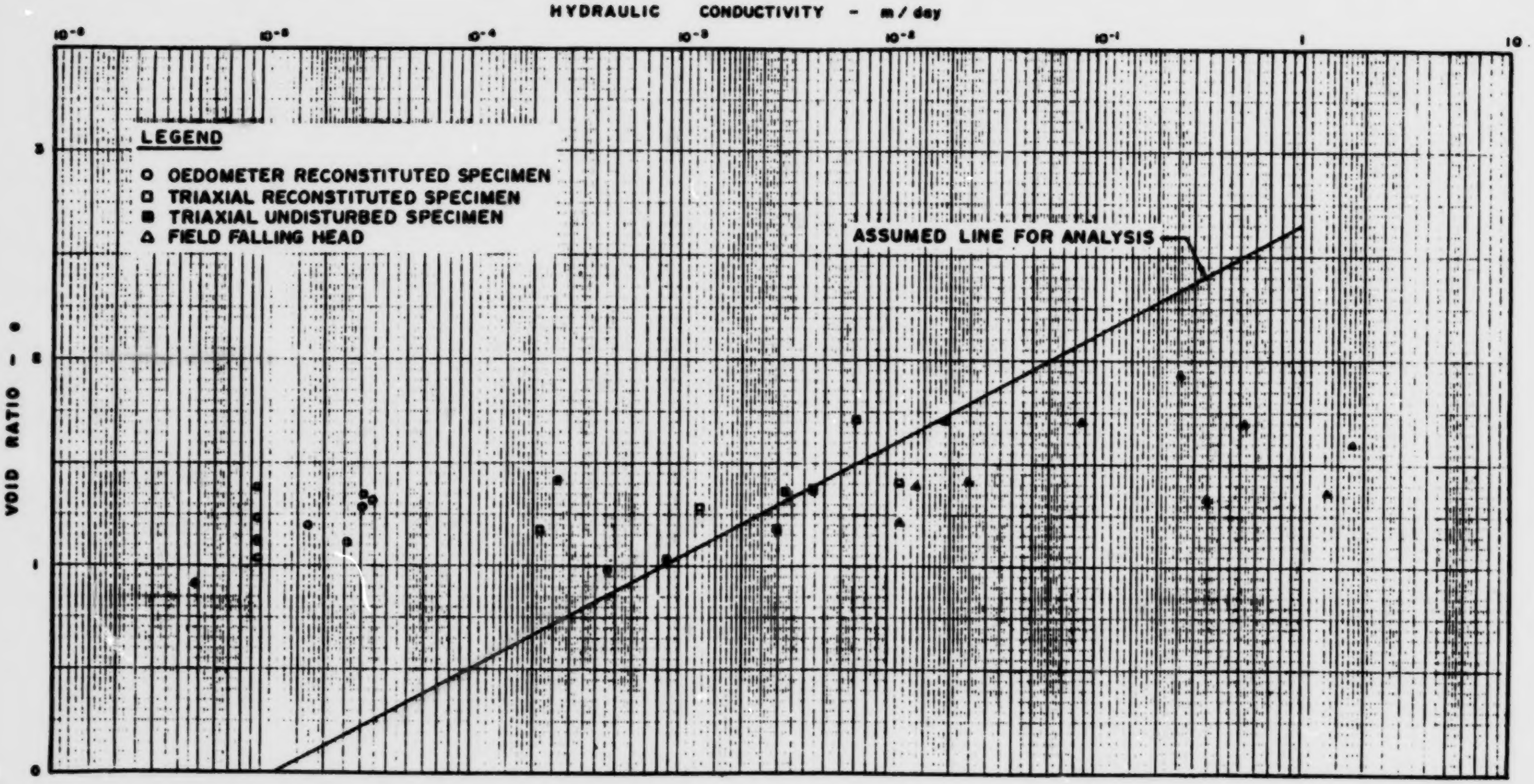


FIGURE 5-20 e-k RELATIONSHIP FROM FIELD AND LABORATORY TEST DATA – ELLIOT LAKE

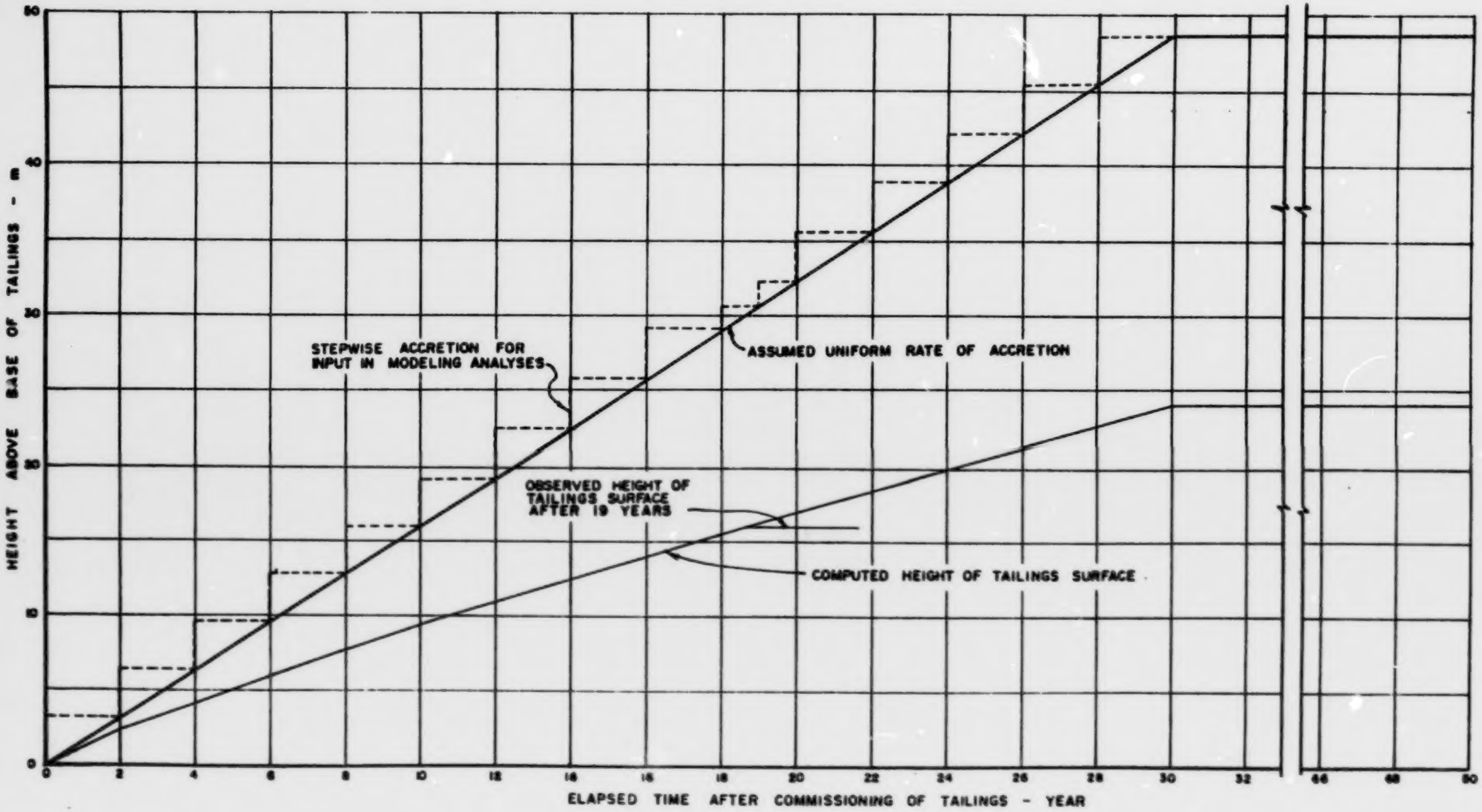


FIGURE 5-21
SELF-WEIGHT CONSOLIDATION AND ACCRETION - ELLIOT LAKE

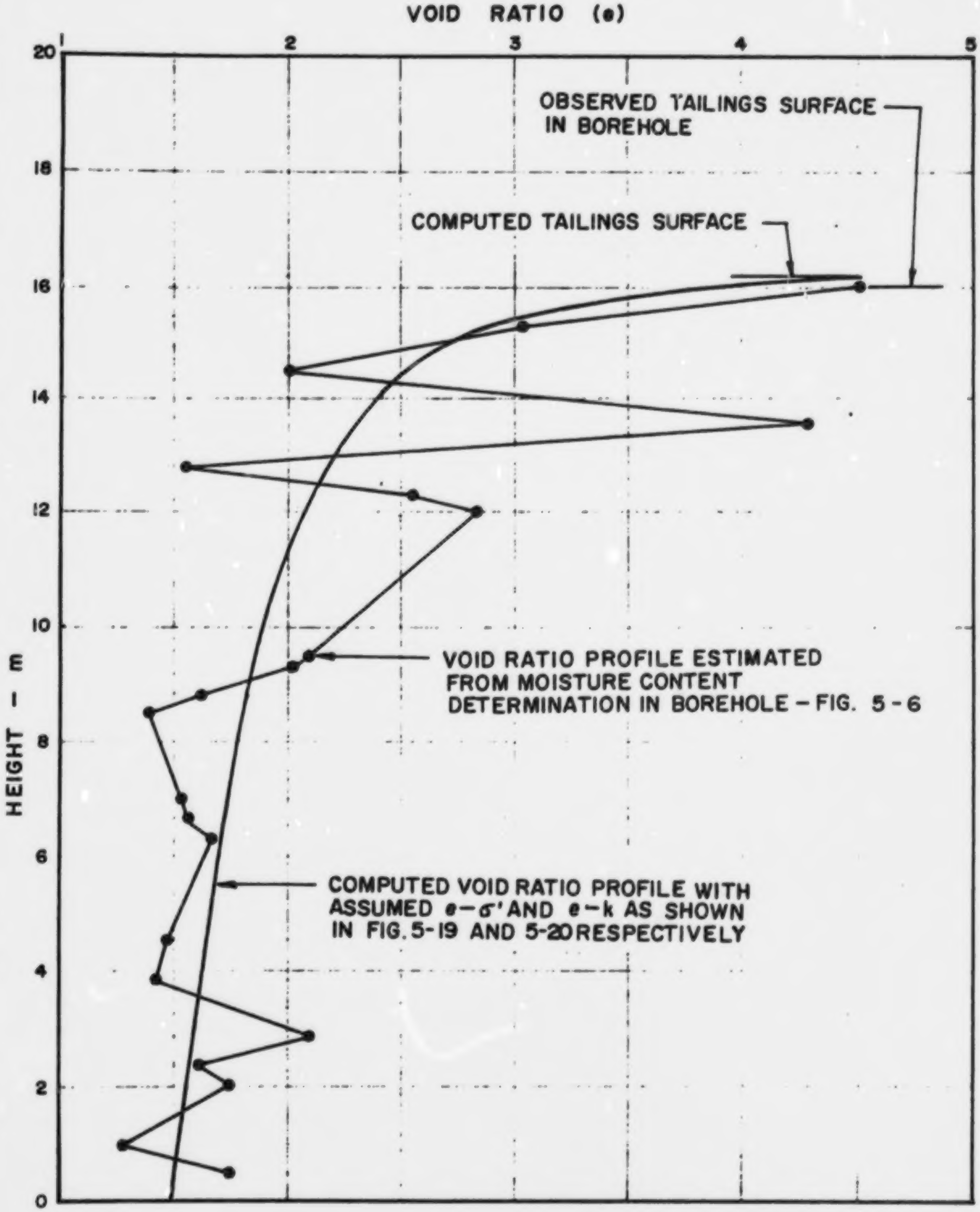


FIGURE 5-22 COMPARISON OF COMPUTED AND FIELD DETERMINED VOID RATIO PROFILES – ELLIOT LAKE

The monthly rates of evaporation used in the analyses were based on an annual total of 810 mm estimated from the Hydrological Atlas of Canada, assumed to be evenly distributed over the months of May through Novermber, i.e. 116 mm per month.

Based on these assumptions, and the methodology described earlier, the post-deposition settlement versus time characteristics of a one dimensional consolidation model without and with a 0.6 m cover, are shown in Figures 5-23 and 5-24, respectively.

Applying the results of the one-dimensional model to the slime predominant clarificaiton pond area, and making assumptions similar to those in the case of the Rabbit Lake project, the estimated settlement profiles of the Elliot Lake model are shown in Figures 5-25 and 5-26 for the case of the tailings without cover, and the case of a 0.6 m cover placed immediately following end of tailings deposition, respectively.

5.3 High Grade Nickel Arsenic Clay Tailings - Key Lake Model

5.3.1 Background Information

The following information has been abstracted from Knight and Haille (1983) and Knight and Piesold Ltd. (1985).

The Key Lake Project is located in northern Saskatchewan, approximately 240 km north of La Ronge on the southern boundary of the Athabasca Sandstone Formation. The area is typified by undulating hills and drumlins, and numerous small lake-filled depressions. Mean annual precipitation and potential evaporation for the site are 460 mm and 508 mm respectively.

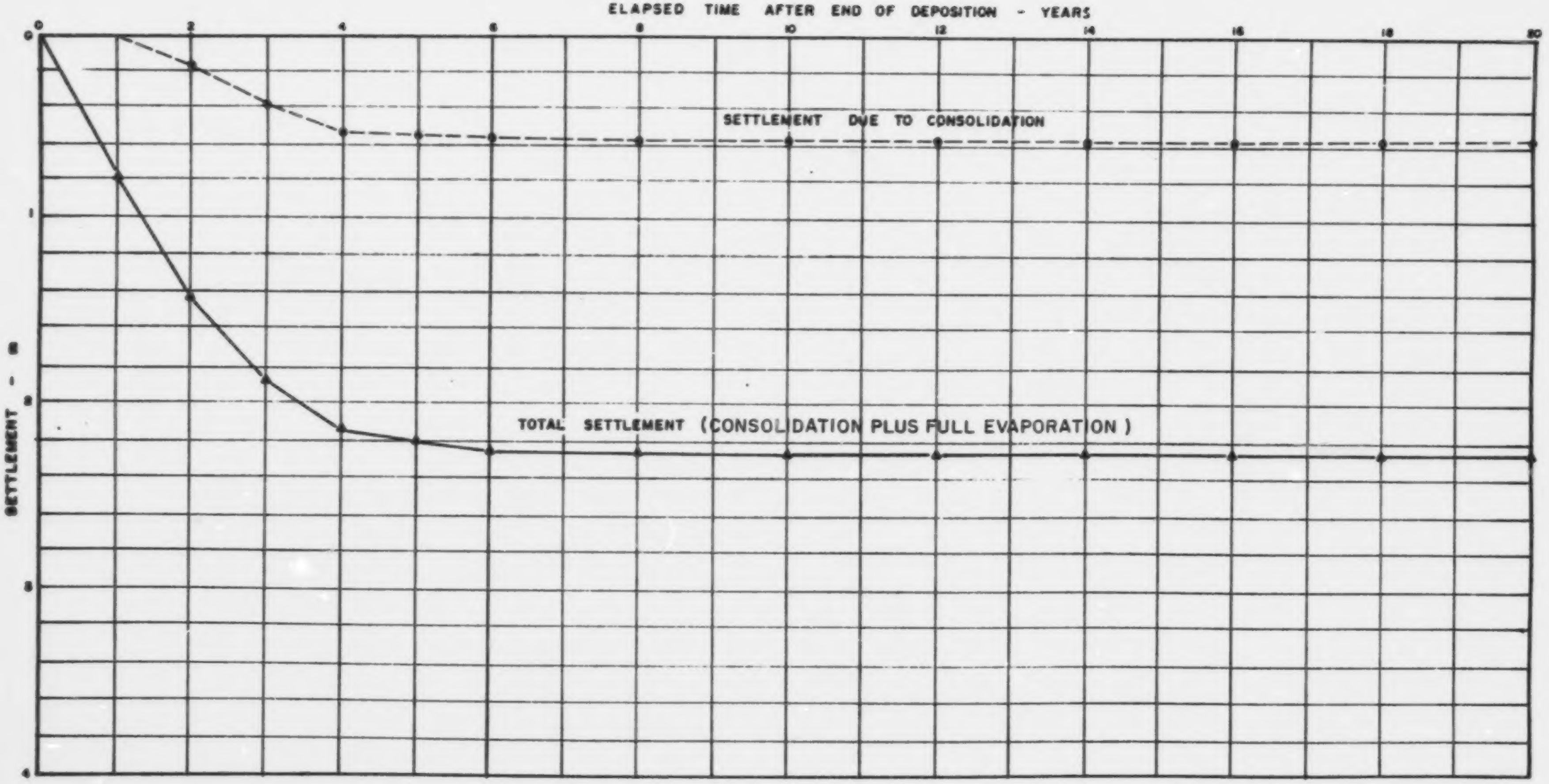


FIGURE 5 - 23
SELF - WEIGHT CONSOLIDATION
FULL EVAPORATION
ELLIOT LAKE MODEL

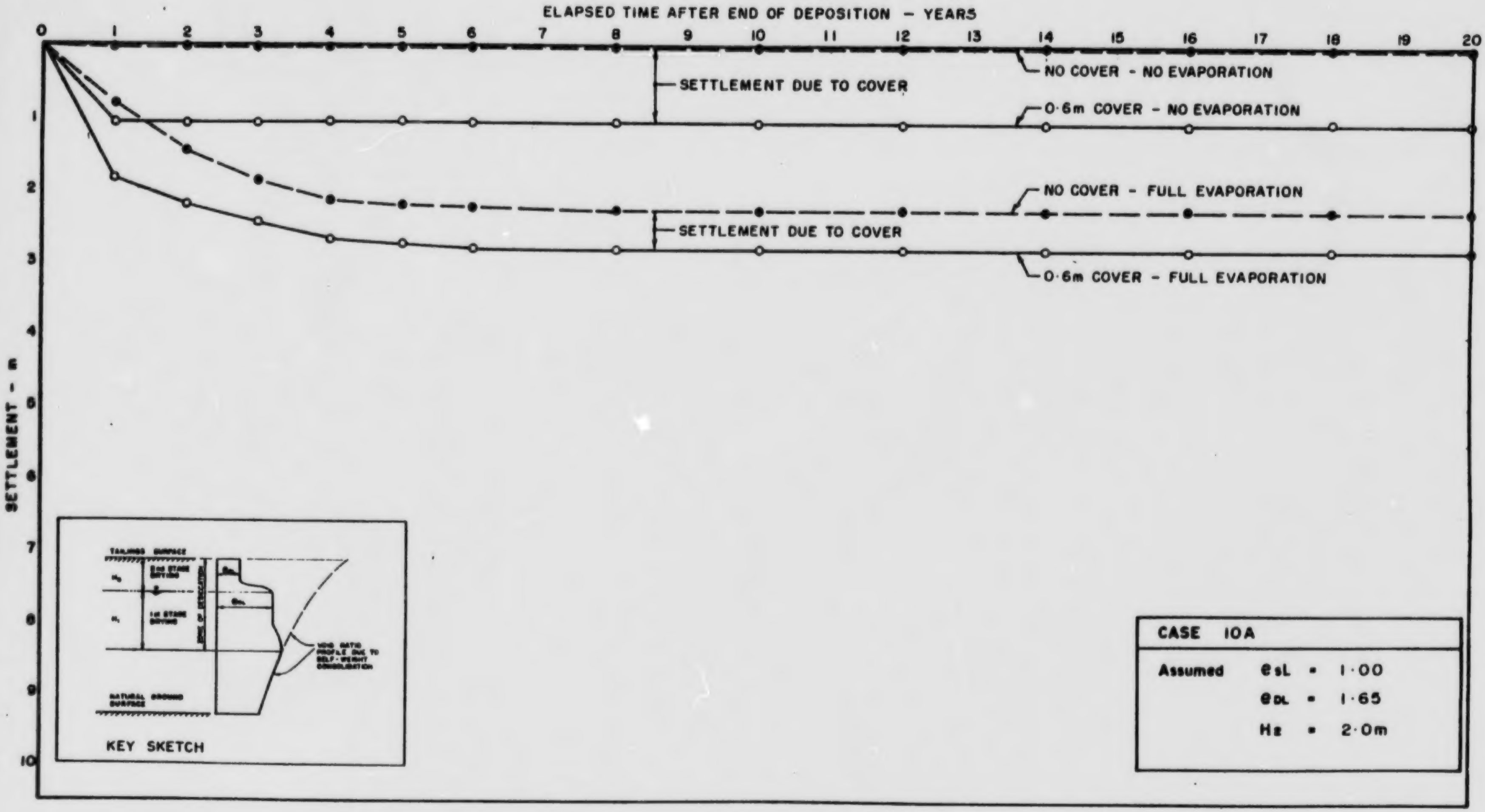


FIGURE 5 - 24

SETTLEMENT OF COVER CONSTRUCTED IMMEDIATELY AFTER END OF TAILINGS DEPOSITION - ELLIOT LAKE MODEL

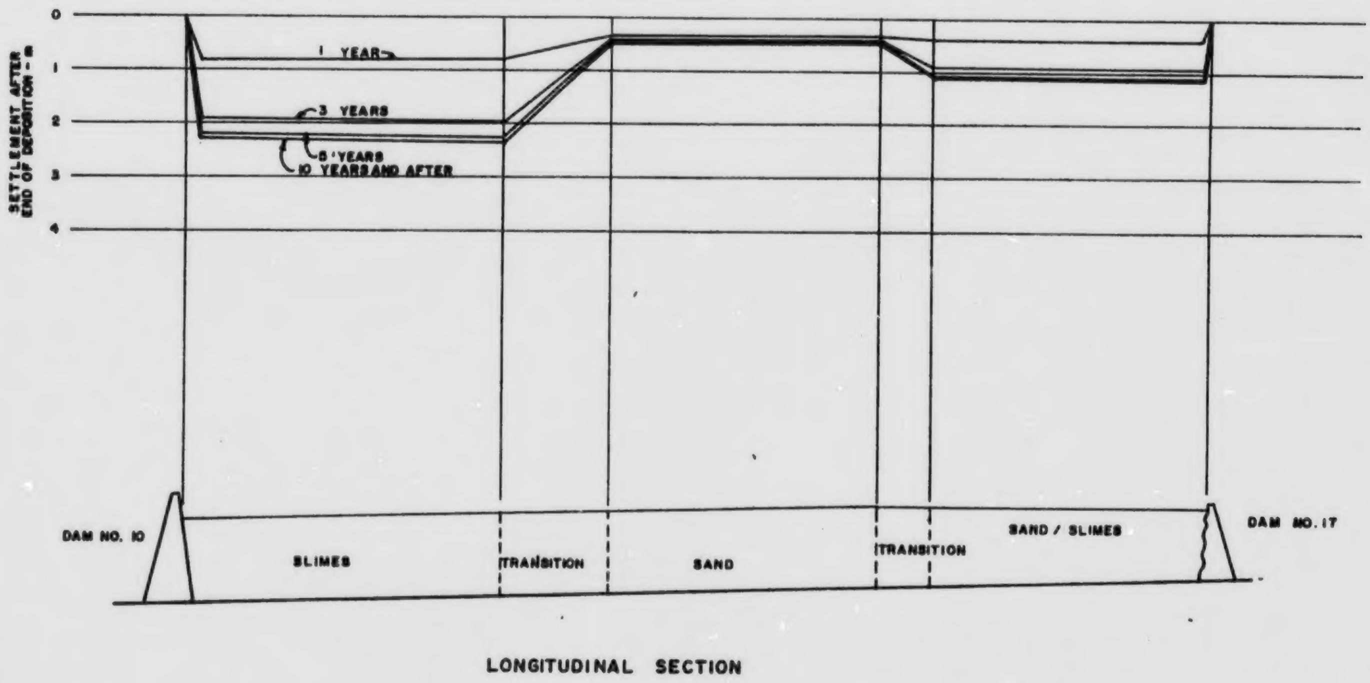
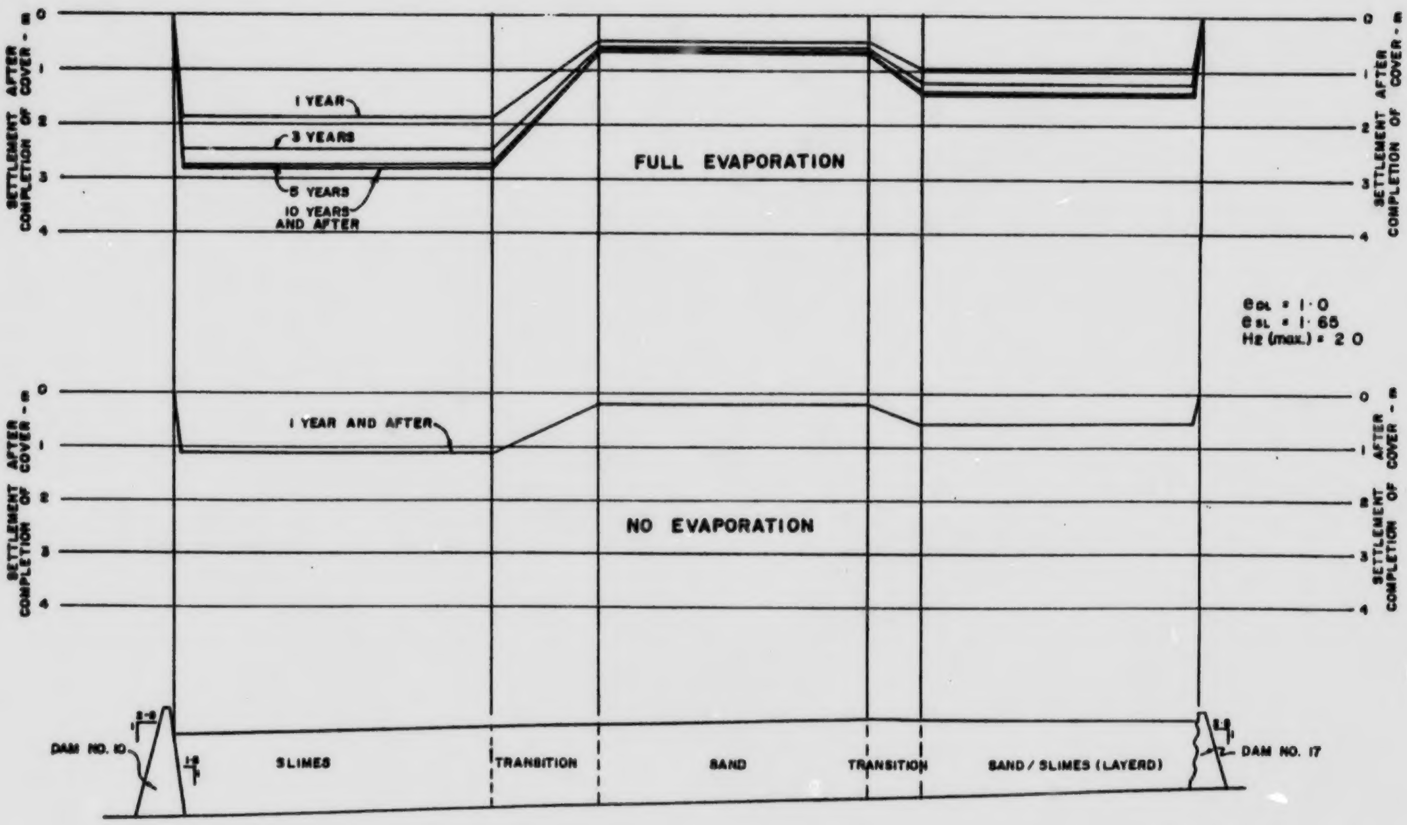


FIGURE 5 - 25
SETTLEMENT PROFILE
FULL EVAPORATION
NO COVER
ELLIOT LAKE MODEL



LONGITUDINAL SECTION

FIGURE 5 - 26
SETTLEMENT PROFILES
COVER CONSTRUCTED IMMEDIATELY
AFTER END OF TAILINGS DEPOSITION

The ore has an average grade of 2.5% U_3O_8 and contains high percentages of nickel and arsenic. The tailings produced from an acid leach process is subsequently neutralized and thickened prior to discharging into the impoundment area, and has the following average characteristics.

Table I
Material parameters for Key Lake tailings

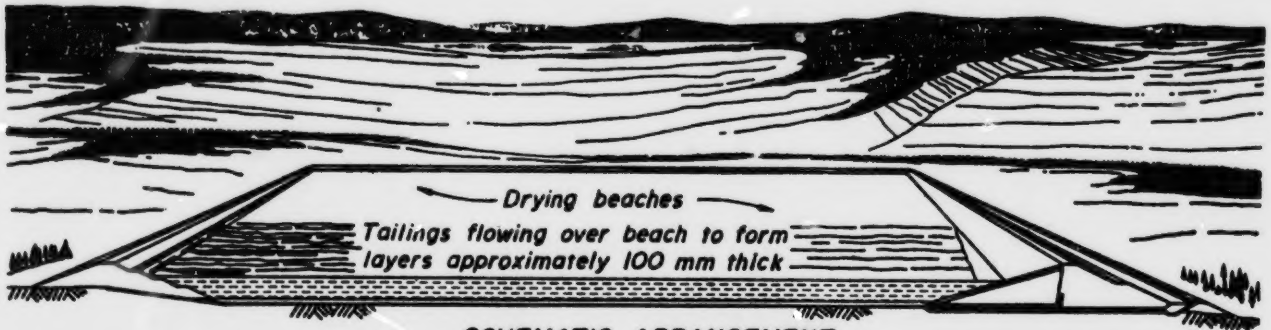
Tailings composition: 70% leach residues, 30% gypsum and metal hydroxides
Solids contents at deposition: 30 - 35% by weight
Temperature at deposition: 40°C
pH at deposition: 10
Average production rate of tailings solids: 1032 tonnes per day
Average unit weight of solids used in design: 8.5 kN/m³
Consolidated undrained triaxial test results:

$c' = 0$
$\phi = 45^\circ$
$c_v = 4.4 \text{ to } 36.6 \text{ m}^2/\text{yr}$
$k \text{ (vertical)} = 7 \times 10^{-8} \text{ m/s}$

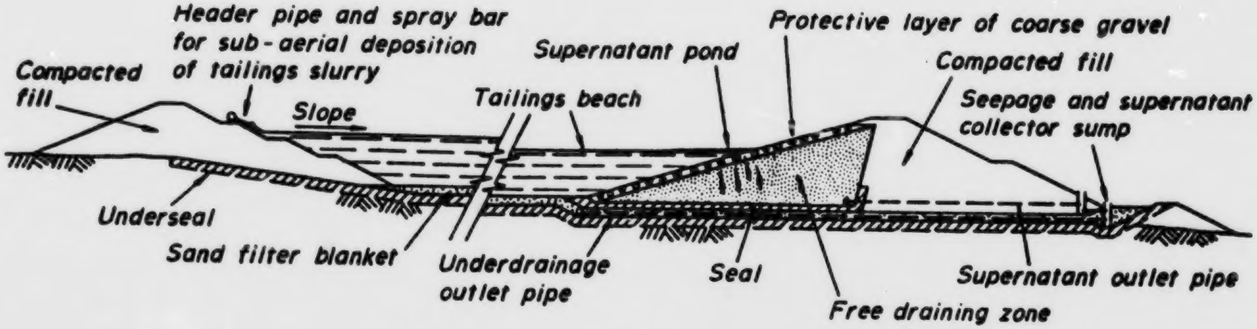
(Knight and Haile, 1983)

The principal feature of the Subaerial Method of tailings deposition involves placing the thickened tailings slurry through spray bars to form thin layers in the range of 100 to 150 mm in thickness. By moving the discharge areas, each layer is allowed to drain and desiccate before the next layer is placed on top. This manner of operation is continued over an eight month period in non-freezing weather. During the remaining four months of sub-zero winter weather, end discharge of the tailings in one continuous thick layer over part of the impoundment area is carried out. This frozen winter-deposited tailings is allowed to thaw, drain and desiccate in summer before resumption of the subaerial deposition in thin layers on top. A schematic of the storage facility is reproduced in Figure 5-27 on the following page.

Recent monitoring of tailings being deposited in the impoundment area reported by Knight and Piesold (1985) indicated in-situ dry densities varying between 0.45 and 0.58 t/m^3 corresponding to solids contents of 35.3


Drying beaches
Tailings flowing over beach to form layers approximately 100 mm thick

SCHEMATIC ARRANGEMENT


Compacted fill
Header pipe and spray bar for sub-aerial deposition of tailings slurry
Slope
Supernatant pond
Tailings beach
Protective layer of coarse gravel
Compacted fill
Seepage and supernatant collector sump
Underseal
Sand filter blanket
Underdrainage outlet pipe
Seal
Free draining zone
Supernatant outlet pipe

SECTION THROUGH EMBANKMENTS

FIGURE 5-27 SCHEMATIC ARRANGEMENT OF TAILINGS STORAGE FACILITY - KEY LAKE PROJECT

(Knight and Haile, 1983)

and 41.7 percent, respectively (or water contents of 183.3 and 139.8 percent, respectively). Settling and air drying tests indicate a limit void ratio corresponding to the saturation limit, e_{SL}, defined by Cargill (1985), of 4.5 with permeability values at this void ratio, ranging between 1.11×10^{-7} and 1.38×10^{-7} m/s determined by constant head permeability tests.

Grain size distribution curves and results of oedometer tests are reproduced in Figures 5-28 and 5-29, respectively on the following pages.

5.3.2 Idealized Model

Based on the general information given in 5.3.1 above, an idealized model was developed in the form of a longitudinal section as shown in Figure 5-34 at the end of this section.

Because of the relatively uniform condition over the plan area of the impoundment, a one-dimensional consolidation model was assumed to be representative of the entire tailings deposit. The assumed e- ' and e-k relationships are shown in Figures 5-30 and 5-31, respectively on the following pages. The rate of accretion was assumed to be 4 m bi-annually with deposition alternating between two halves of the impoundment to allow thawing and subsequent drainage and desiccation of the tailings deposited in winter. This is equivalent to an average annual accretion of 2 m. The intermittent manner of accretion is shown in Figure 5-32. Initial void ratio of the settled tailings was assumed to be 5.5, the saturation limit (e_{SL}) was assumed 4.5 and the desiccation limit (e_{DL}) was assumed to be 2.0. Other parametric values used are:

S = 75%
C'_E = 1.0
C_D = 0.9
H_2 (max) = 2.0 m

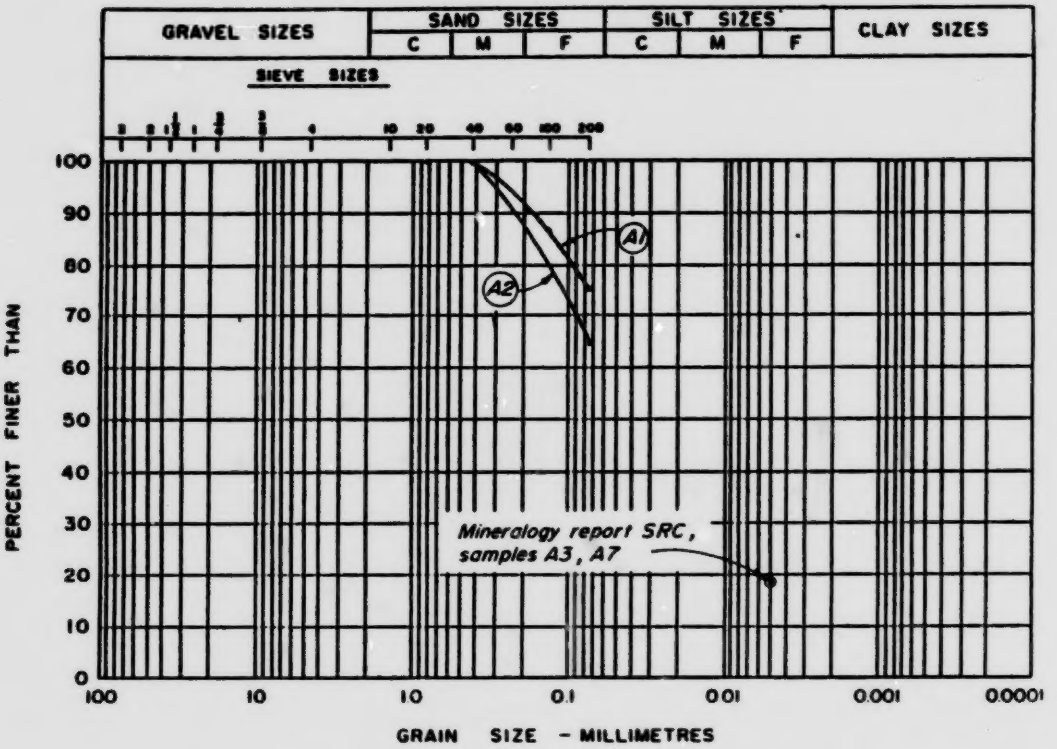
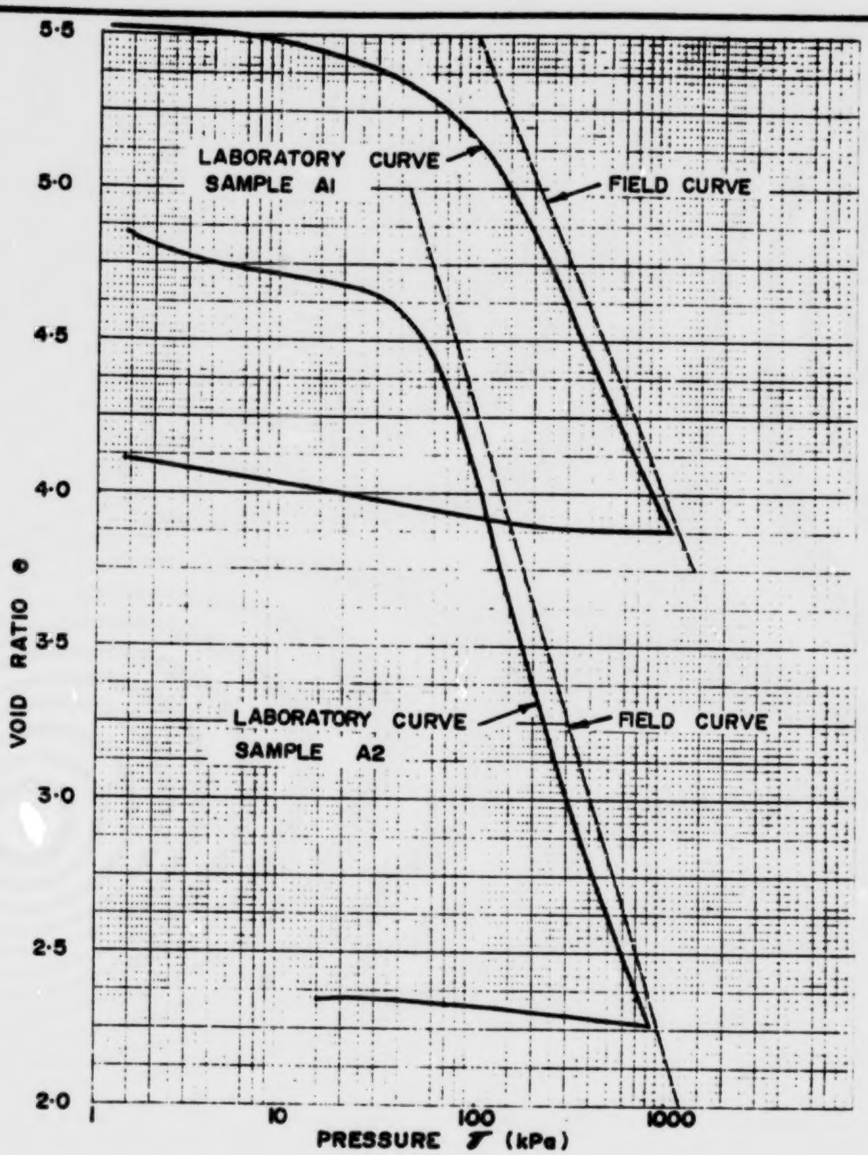


FIGURE 5-28 GRAIN SIZE DISTRIBUTION
KEY LAKE PROJECT

(KNIGHT AND PIESOLD, 1985)



SAMPLE	STRESS RANGE (kPa)	c_v (m^2/yr)	m_v (m^2/kN)	CALCULATED k (m/s)
A1	240			
		52.73	.00031	5×10^{-9}
	495			
		102.00	.00015	4×10^{-9}
	1017			
A2	56			
		37.70	.00194	2.2×10^{-8}
	98.5			
		64.91	.00143	2.8×10^{-8}
	197			
		41.25	.00059	7×10^{-9}
	422			
		15.47	.00034	1×10^{-9}
	816			

c_v = coefficient of consolidation
m_v = coefficient of compressibility
k = coefficient of permeability

FIGURE 5-29 OEDOMETER TEST RESULTS
KEY LAKE PROJECT
(KNIGHT AND PIESOLD, 1985)

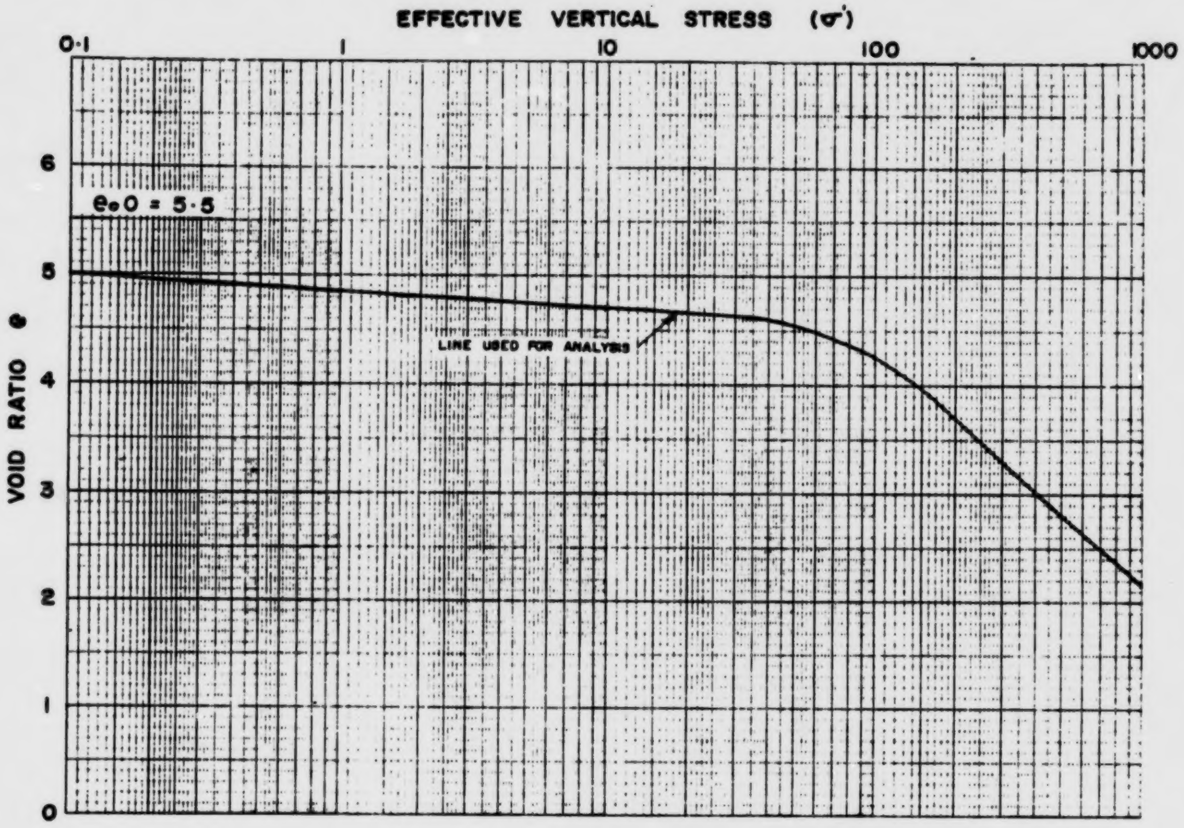
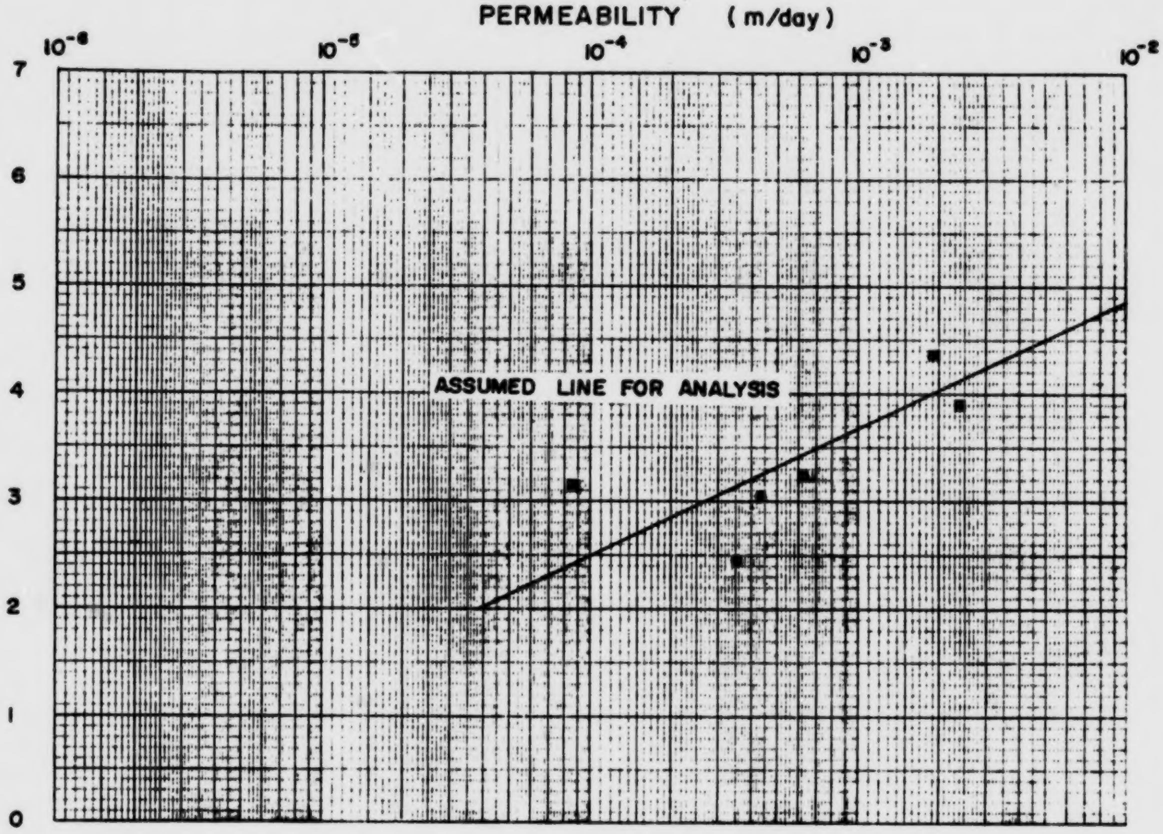


FIGURE 5-30 e - σ' RELATIONSHIP FROM FIELD AND LABORATORY TEST DATA - KEY LAKE PROJECT



LEGEND
- ● OEDOMETER TEST A1
- ■ OEDOMETER TEST A2

FIGURE 5-31 e - k RELATIONSHIP FROM FIELD AND LABORATORY TEST-DATA - KEY LAKE PROJECT

The monthly rates of evaporation and precipitation recorded at Key Lake over an 11-year period obtained from Environment Canada are given below:

Month	Evaporation - mm	Precipitation - mm
January	0	14.8
February	0	15.4
March	0	15.6
April	0	21.7
May	109.9	25.8
June	112.3	50.8
July	120.2	79.1
August	91.8	60.5
September	47.6	57.3
October	0	30.1
November	0	30.1
December	0	23.8

The self-weight consolidation and desiccation effects during deposition over a 13-year period and a post-deposition period of 4 years were modelled based on the above assumptions and the results shown in Figure 5-32. As anticipated from the manner of deposition, consolidation settlements throughout the deposition and post-deposition periods are controlled by desiccation effects as reflected by the computed void ratio profile at the end of deposition shown in Figure 5-33. However, due to the relatively limited potential evaporation at the site, the tailings is essentially fully saturated throughout except very thin zones which are indicated to be desiccated to below the saturation limit, e_{SL}.

Because of the desiccated nature of the tailings, and relatively low stresses due to placement of a thin earth/rip-rap cover, the settlement due to cover load alone can be expected to be small and long term cover settlements

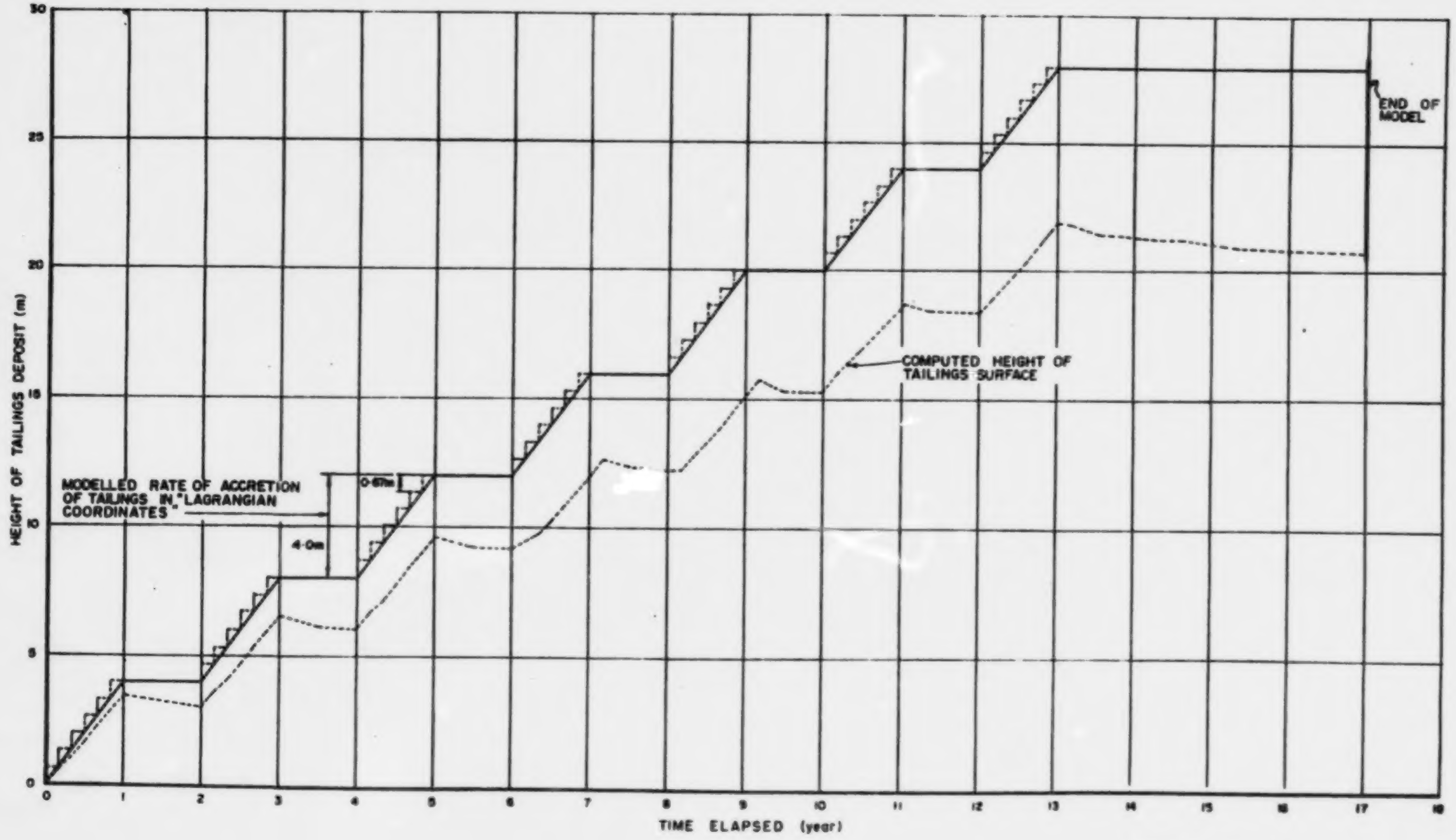


FIGURE 5 - 32
SELF - WEIGHT CONSOLIDATION
AND ACCRETION - KEY LAKE

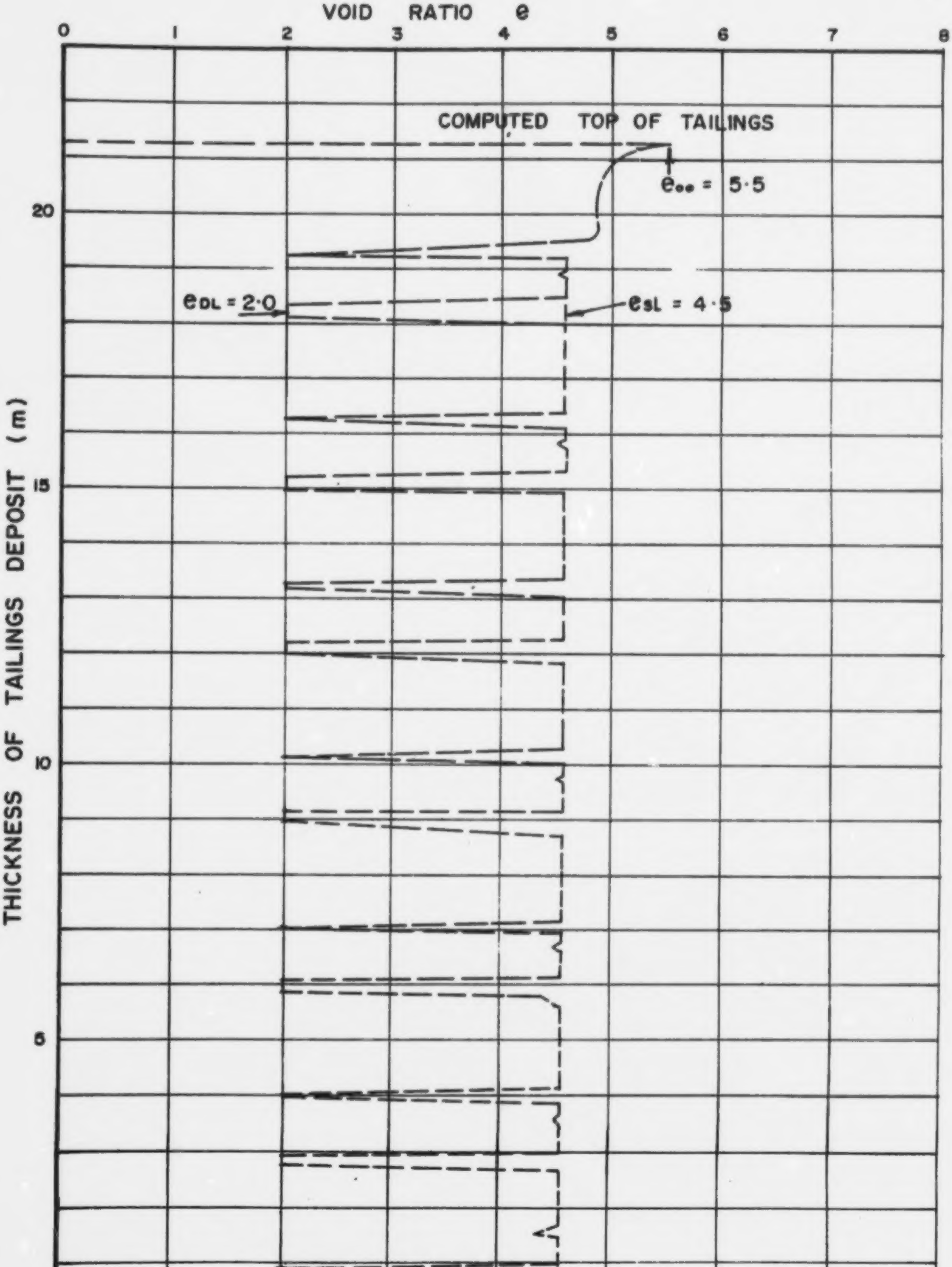
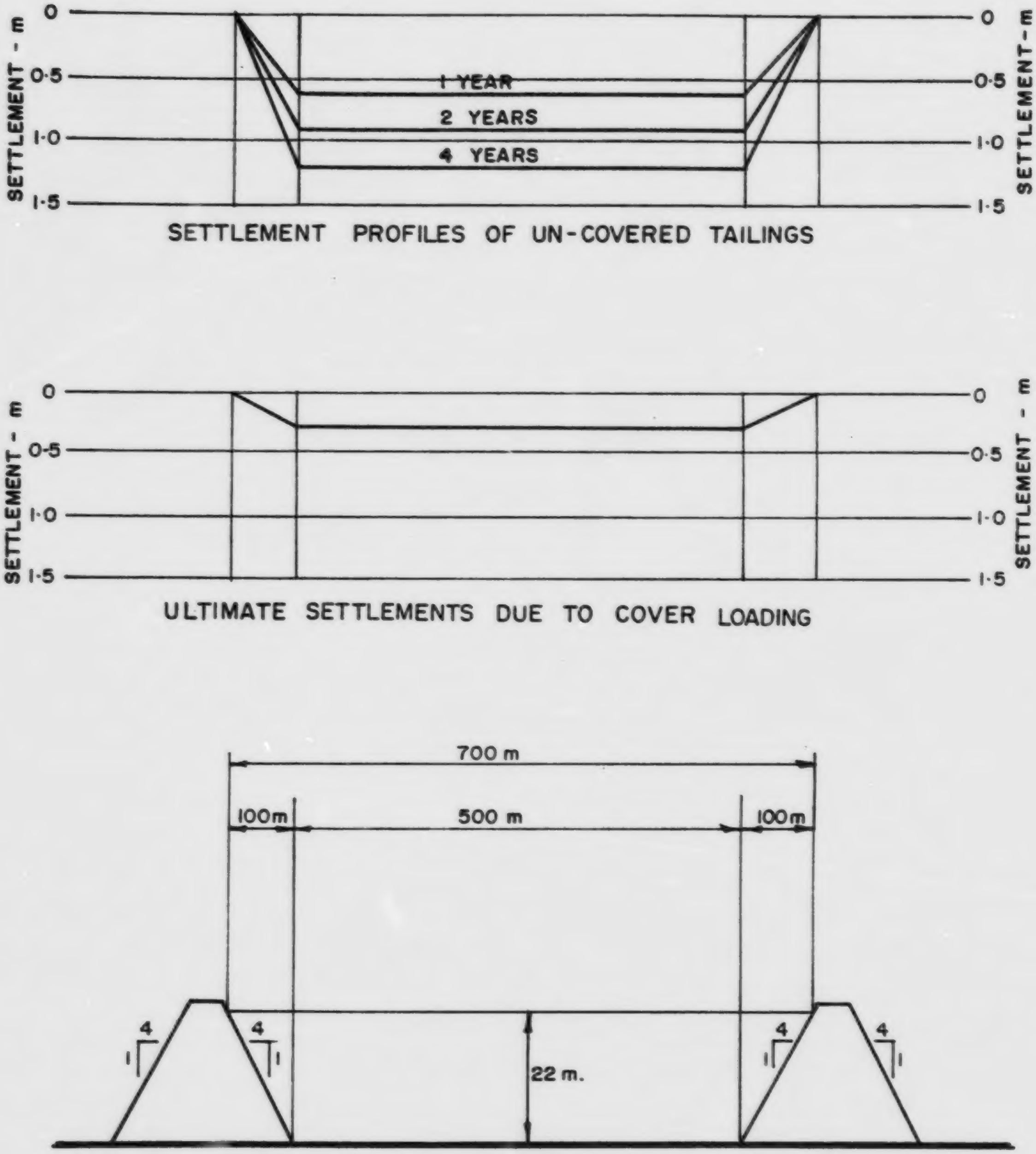


FIGURE 5 - 33 VOID RATIO PROFILE AT END OF TAILINGS DEPOSITION - KEY LAKE MODEL

will be controlled by desiccation effects. Based on the $e - \sigma'$ relationship shown in Figure 5-30 and the loading from a 0.6 m thick earth/ rip-rap cover, it was estimated that an ultimate settlement of 0.28 m would occur over the long term due to cover loading alone. Ongoing settlements after the end of deposition due to desiccation as indicated in Figure 5-32 are considerably more prominent compared to this settlement due to cover loading. In fact, if desiccation of the tailings throughout its full depth to the assumed desiccation limit, e_{DL} could take place in the long term, an ultimate settlement exceeding 8 m could occur theoretically. In reality of course, the actual maximum depth of desiccation will be limited by the capillary potential of the tailings and any surface controls such as cracking and cover design. Simplistic settlement profiles of the tailings surface due to desiccation effects at various periods after the end of deposition and the ultimate settlement due to cover loading alone are shown in Figure 5-34 for illustrative purposes.



SETTLEMENT PROFILES OF UN-COVERED TAILINGS

ULTIMATE SETTLEMENTS DUE TO COVER LOADING

FIGURE 5-34 SETTLEMENT PROFILES - KEY LAKE PROJECT

6.0 IDENTIFICATION OF LONG TERM PROBLEMS RELATED TO CONSOLIDATION

The original terms of reference of this study require that "the problems resulting from slime-sand segregation and low consolidation as they relate to long-term environmental aspects of uranium tailings" be defined.

Slime-sand segregation is associated with conventional hydraulic deposition of tailings as discussed earlier and the cumulation of slimes deposited in a loose state within the clarification pond area is of principal concern related to long term environmental impact. It is important to note, however, that the loose state or "low consolidation" of the segregated slimes in itself, may not be a serious problem. The results of modelling indicated that in fact high to very high degrees of consolidation under self-weight of the tailings could be achieved during the progress of deposition as a result of relatively slow rate of accretion. For example in the Rabbit Lake case, a computed degree of consolidation of 97.9 percent at the end of tailings deposition was obtained (page 4-15), which is supported by actual field observations from piezometers (Figure 4-4). Similar observations were also indicated by Matyas et al (1984) for Elliot Lake tailings. Although some residual settlements under the self-weight of the tailings would still occur in the long term (for example, 0.41 m in the Rabbit Lake case indicated on page 4-15), no significant problems are anticipated from such settlements. Similarly, post-construction settlements due to the loading from a thin soil/rip-rap cover would also be relatively tolerable and manageable. For example, in the Rabbit Lake case, a maximum settlement of about 1 m would be anticipated due to the loading from a 0.6 m thick cover placed immediately after the end of tailings deposition before desiccation of the tailings can take place (Fig. 4-11A). The greater part of this settlement would also be completed within two to three years after construction so that the long term impact due to cover loading alone is relatively negligible.

The loose initial state of the deposited tailings, particularly the part near its surface, however, does provide a favourable condition for large settlements induced by other mechanisms such as desiccation or lowering of the water level within the tailings as discussed below.

Desiccation due to evaporation, is a considerably more significant process related to large long term consolidation settlements of segregated slimes in the clarification pond area. This is a natural consequence of the time dependent nature of the desiccation process itself. Referring to Figures 4-7A through 4-7D, it can be seen that, depending on the maximum depth of desiccation, lowering of the water level within the tailings could take place over a period of more than 10 years. This is accompanied by shrinkage of the tailings above the water level and an increasing surcharge loading on the tailings below, which are manifested as large long term settlements of the tailings surface if desiccation is allowed to take place to significant depths. The sensitivity of long term settlement of the tailings surface to the depth of desiccation and other parameters related to the process, is highlighted in Figure 4-10. Prediction of the depth of desiccation or lowering of the water level within the tailings is actually a very complex problem involving a large number of variables (Ferguson, Hutchison and Schiffman, 1985). Simplification of the problem in the method by Cargill (1985) used in the present study may not be valid in all cases. The present state-of-the-knowledge regarding the empirical parameters (e_{SL} and e_{DL}) for uranium tailings, which are required in predicting desiccation settlements, is also very limited. Therefore, prediction of the progress of consolidation in the long term is still fraught with uncertainties, and thus assessment of its impact on environmental issues has to rely on probable ranges of conditions rather than definitive predicitons.

The implications of large settlements and slow rate of consolidation, resulting from slime-sand segregation and low consolidation, are of course mainly in connection with disturbances to surface drainage measures resulting in increased infiltration of surface water into the tailings, and consequently sustained or increased rate of release of contaminated interstitial water through seepages to the groundwater environment.

From the general discussion given earlier in Section 2.0, it is evident that cold weather could also play an important part in the long term performance of a tailings deposit. Freezing of tailings deposited in slurry form traps a large proportion of contaminated water within the frozen mass. If the tailings remains frozen in the long term, its surface will not settle, thus accentuating the differential settlement in the transition zone between the clarification pond area and the beach areas. If however, the tailings thaw out in the long term due to climatic changes or other natural processes, the excess interstitial water will be released to the environment at rates governed by both thawing and consolidation, and large surface settlement will occur which could damage the surface drainage measures accompanied by increased infiltration and release of contaminated water.

7.0 METHODS TO REDUCE SEGREGATION AND PROMOTE CONSOLIDATION OF NEW TAILINGS DEPOSITS

With conventional types of hydraulic deposition such as those employed at Elliot Lake for the low-grade pyritic type of tailings and at Rabbit Lake for medium-grade neutral type of tailings, slime-sand segregation will occur as a natural transport-sedimentation process which cannot be prevented. To reduce the effects of areal segregation, it is theoretically feasible to relocate the discharge points and clarification pond to achieve more-or-less uniform overall distribution of slimes over the entire impoundment area to minimize long term differential settlements. However, practical requirements for strategic location of decant works and minimization of dam and other forms of construction would certainly severely limit the economic and operational viability of such a scheme.

Thickened tailings slurry, on the other hand will reduce to a certain degree, segregation on deposition due to hindered sedimentation effects. The most effective method to practically eliminate segregation is by predewatering the tailings to a semi-solid state by vacuum or pressure filters such as the system currently employed in the Collins Bay B-Zone project. In addition to eliminating segregation, removal of a large part of the free water also preconsolidates the tailings and thus minimizes long term consolidation effects. This method is of course associated with high capital and operating costs which are offset by benefits from reduction in volume of tailings to be stored and other process related benefits. A detailed analysis of costs and benefits for such systems is beyond the scope of this study. A thorough review of available filter dewatering equipment along with other dewatering methods including thickening, cycloning, chemical treatment, thermal dewatering, electrokinetic methods and so forth, is given by Roe (1979) together with their advantages and disadvantages.

A discussion of the use of various forms of internal drainage systems to promote in-situ dewatering and thus consolidation of uranium tailings was given by Charlie and Martin (1980). The systems considered include finger and blanket drains and planned segregation by using spigots or cyclones to create internal drainage zones. Illustrations of the various systems given by the authors are reproduced below.

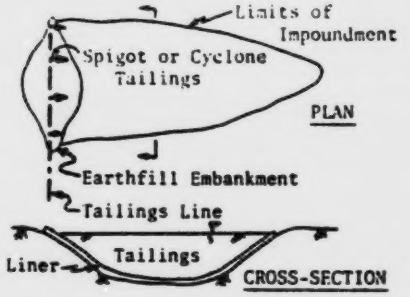

Limits of Impoundment
Spigot or Cyclone Tailings
PLAN
Earthfill Embankment
Tailings Line
Tailings
Liner
CROSS-SECTION

Fig. 7. Typical existing impoundment without internal drainage.

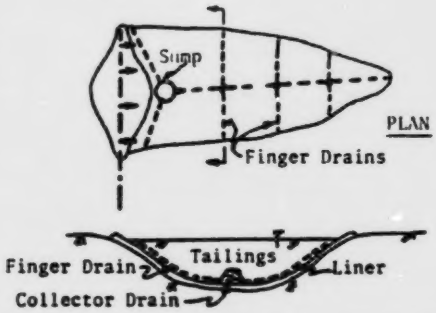

Sump
PLAN
Finger Drains
Tailings
Finger Drain
Liner
Collector Drain

Fig. 8. Interior drainage system utilizing finger drains.

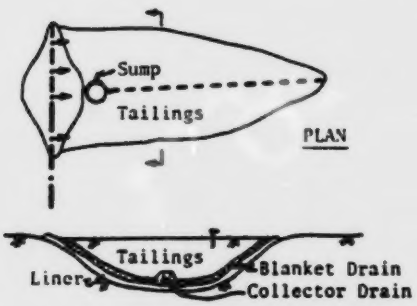

Sump
Tailings
PLAN
Tailings
Liner
Blanket Drain
Collector Drain

Fig. 9, Interior drainage utilizing a blanket drain.

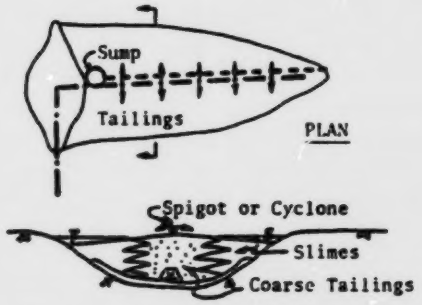

Sump
Tailings
PLAN
Spigot or Cyclone
Slimes
Coarse Tailings

Fig. 10. Interior drainage utilizing tailings segregation.

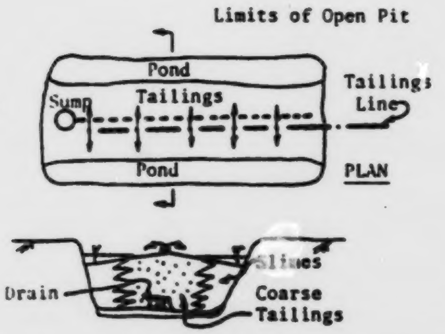

Limits of Open Pit
Pond
Tailings
Sump
Tailings Line
Pond
PLAN
Slimes
Drain
Coarse Tailings

Fig. 11. Open pit drainage utilizing tailings segregation.

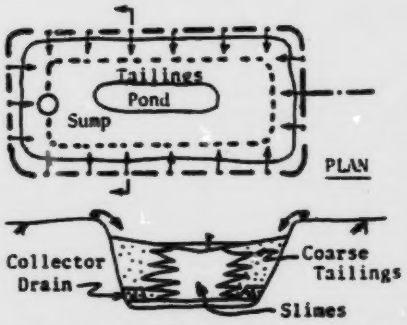

Tailings
Pond
Sump
PLAN
Collector Drain
Coarse Tailings
Slimes

Fig. 12. Interior drainage utilizing segregation and capillary break.

These measures are designed to promote the rate of consolidation under self-weight of the tailings during deposition. However, as indicated earlier in 6.0, modelling and actual field observations indicated that high degrees of consolidation were generally achieved in existing conventional uranium tailings impoundments in Canada and thus will be achievable in similar new facilities without such special internal drainage measures. Guided by such findings, it is believed that it will be much more cost effective to focus attention on post-deposition treatments and strategic design and planning of cover construction to take advantage of desiccation effects and thus minimize long term consolidation due to conditions imposed on the tailings after end of deposition. Such measures are applicable to both new and existing tailings deposits and are described in Section 8.0 below.

8.0 METHODS TO PROMOTE CONSOLIDATION OF EXISTING TAILINGS DEPOSITS

The objectives of promoting consolidation of tailings are two-fold:

i) to facilitate access to the tailings surface during cover construction

ii) to minimize post-construction consolidation of the tailings and settlement of the cover.

The findings of the modelling study carried out in this study suggest that a simple and cost effective method could be developed to achieve the above goals by taking advantage of desiccation effects. Exploitation of evaporative desiccation to achieve similar goals by special operational procedures during tailings deposition is already applied in the case of the subaerial method in the Key Lake project and has been considered as an effective means for dewatering of dredged materials (Krizek, Giger and Jin, 1975). A method specially suited to deal with conventional hydraulically deposited uranium tailings similar to the Rabbit Lake case is described in Section 8.1 below. Other available but less cost effective methods are described later in Section 8.2.

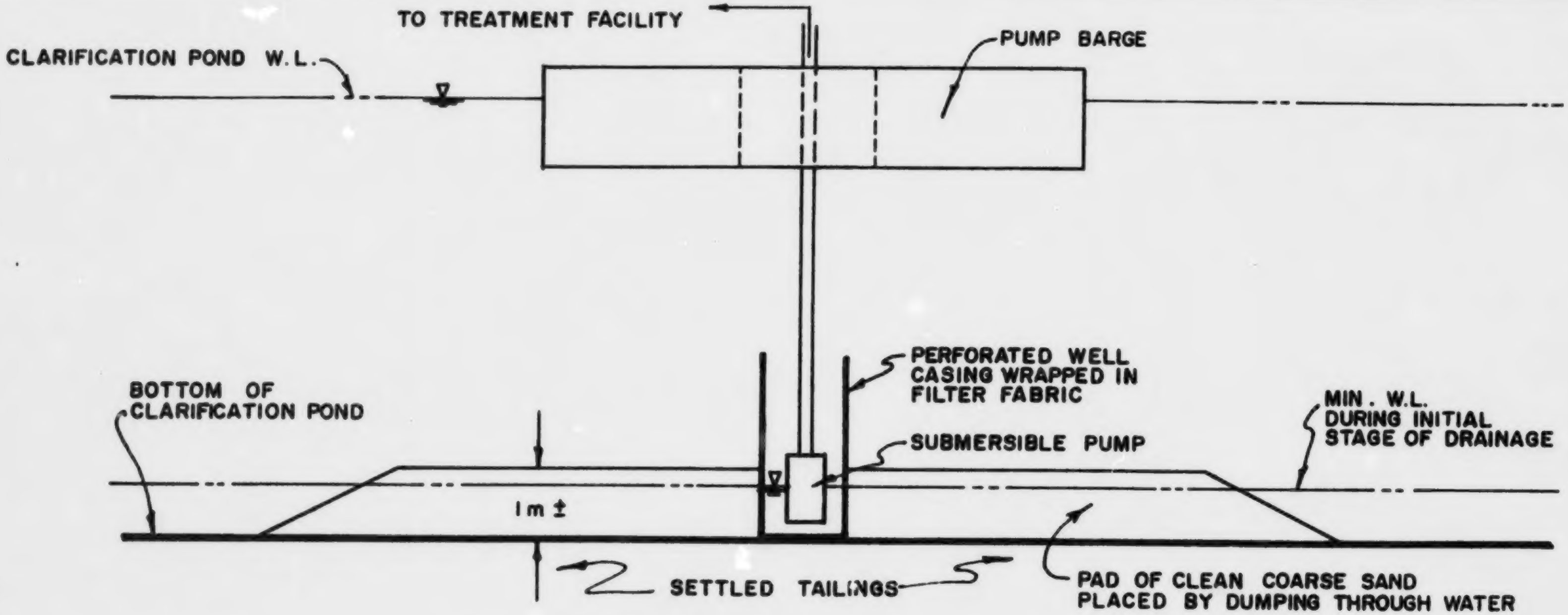
8.1 Deferred Cover Construction and Surcharging

To take full advantage of desiccation effects particularly on the slimes within the clarification pond area, it is obvious that the first step to be taken is to completely drain the water within the clarification pond as soon as possible after completion of tailings deposition. The method of drainage will depend on the original decant arrangement. If a pumped decant system was used, continued pumping from a screened well as shown in

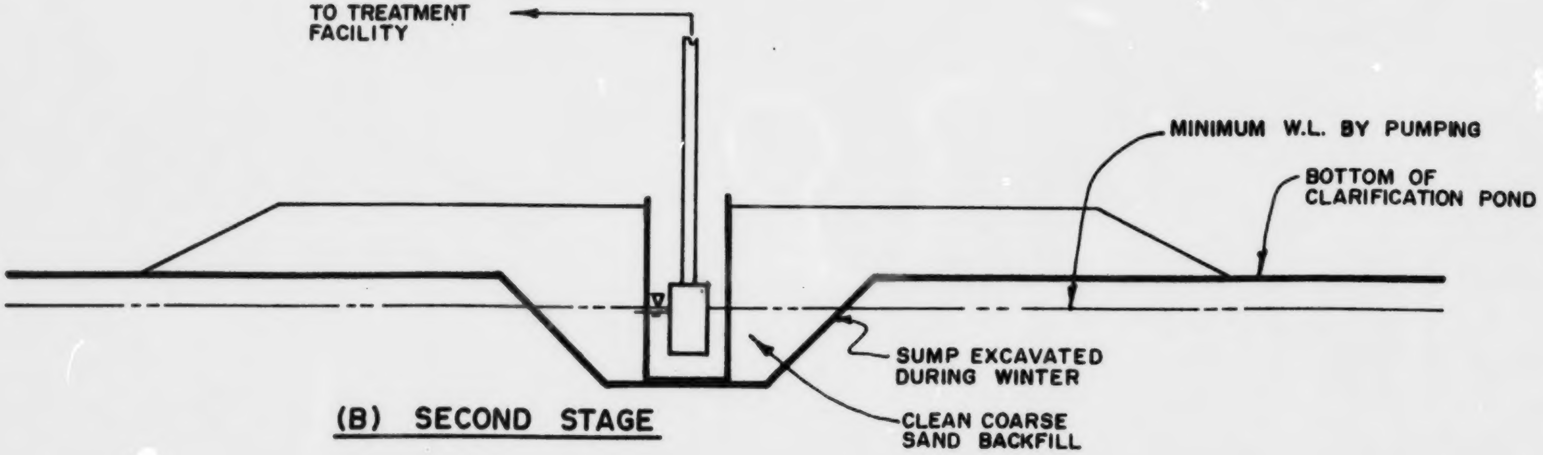
Figure 8-1 to prevent drawing off the loose tailings at the bottom of the pond, will enable effective dewatering of the pond without major construction efforts. Location of the pump well at this point takes full advantage of the natural shape of the tailings surface developed during deposition, thereby allowing surface runoff to converge towards the well without the need to construct any special system of collection ditches. During freeze up in the first winter of operation, deepening of the pump well could be readily accomplished by excavating a sump and lowering the pump well as shown in Figure 8-1(B). This will enable complete drainage of the clarification pond. If desired, finger drains radiating from the pump well could also be constructed during this stage, by excavating trenches in the frozen tailings surface and backfilling with clean coarse sand, to promote drainage in the upper part of the tailings.

A similar approach could be taken to drain the clarification pond in the case where a decant weir or tower is used. This is shown schematically in Figure 8-2.

By providing effective surface drainage in the manner as described above and allowing desiccation to take place over a suitable period of time, recession of the water level to a sufficient depth below the surface will take place to provide a trafficable surface to enable construction of the cover to proceed. The actual period required for this purpose will depend on the desiccation properties of the tailings and site specific meteorological conditions. For discussion purposes herein, however, assuming the case shown in Figure 4-7C for the Rabbit Lake project, after a period of say 3 years, the water level would have receded to more than 1 m below the surface of the tailings due to evaporative desiccation. At the same time about 1.6 m of settlement of the tailings surface would have occurred,

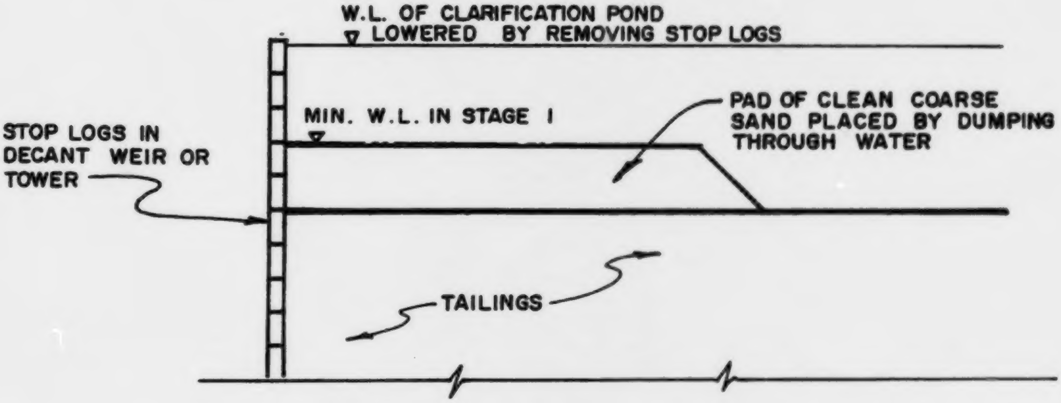

TO TREATMENT FACILITY
CLARIFICATION POND W.L.
PUMP BARGE
BOTTOM OF CLARIFICATION POND
PERFORATED WELL CASING WRAPPED IN FILTER FABRIC
SUBMERSIBLE PUMP
MIN. W.L. DURING INITIAL STAGE OF DRAINAGE
1 m ±
SETTLED TAILINGS
PAD OF CLEAN COARSE SAND PLACED BY DUMPING THROUGH WATER

(A) INITIAL STAGE

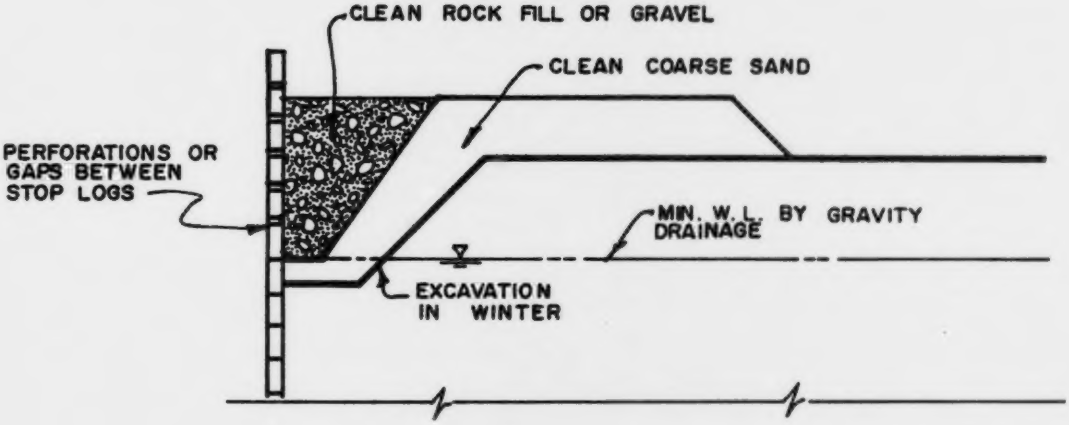

TO TREATMENT FACILITY
MINIMUM W.L. BY PUMPING
BOTTOM OF CLARIFICATION POND
SUMP EXCAVATED DURING WINTER
CLEAN COARSE SAND BACKFILL

(B) SECOND STAGE

FIGURE 8 - 1 DRAINAGE OF CLARIFICATION POND BY PUMPING


W.L. OF CLARIFICATION POND
LOWERED BY REMOVING STOP LOGS
MIN. W.L. IN STAGE I
PAD OF CLEAN COARSE SAND PLACED BY DUMPING THROUGH WATER
STOP LOGS IN DECANT WEIR OR TOWER
TAILINGS

(A) STAGE I


CLEAN ROCK FILL OR GRAVEL
CLEAN COARSE SAND
PERFORATIONS OR GAPS BETWEEN STOP LOGS
MIN. W.L. BY GRAVITY DRAINAGE
EXCAVATION IN WINTER

(B) STAGE 2

FIGURE 8-2 GRAVITY DRAINAGE OF CLARIFICATION POND THROUGH DECANT WEIR OR TOWER

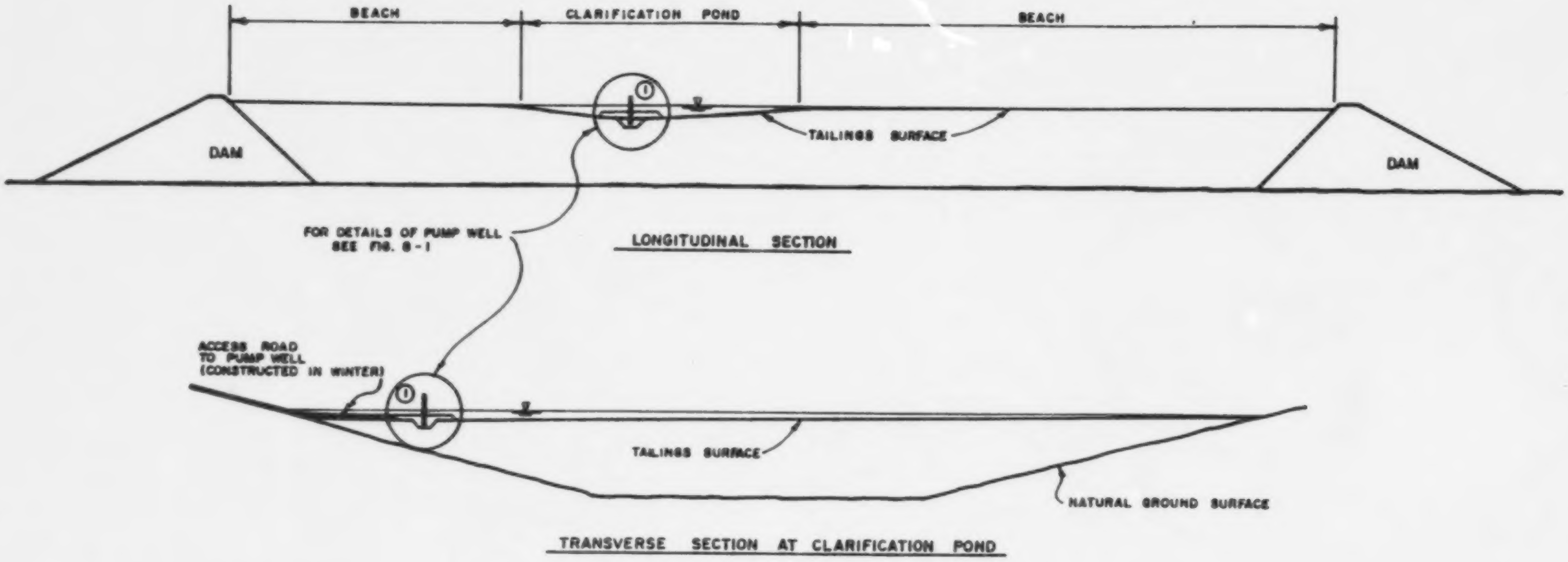
representing over 70 percent of the ultimate settlement due to self-weight and desiccation effects. If a 0.6 m thick cover is placed at that time, a computed range of post-construction settlements between 0.6 m and 1.2 m shown in Figure 4-11C would occur over the long term depending on whether the cover is fully effective or non-effective in preventing further evaporation from the tailing. This deferment of cover construction results in a reduction of post-construction settlements of some 50 percent when compared with the range of settlements for the case of cover construction carried out immediately following completion of tailings deposition, which is also shown in Figure 4-11C. Further reduction in post-construction settlements could of course be achieved if so desired by deferring construction of the cover for a longer period of time, and minimization of settlements could also be achieved by designing the cover as an effective evaporation barrier.

If it is desirable to reduce the time to complete cover construction, a scheme of combined surface drainage, desiccation and strategic surcharging could be used to pre-induce consolidation and settlement prior to finishing the cover. The basic approach and sequence of operations are shown schematically in Figures 8-3A, 8-3B and 8-3C, using the general conditions at the Rabbit Lake Mine tailings area for illustration purposes. The principal steps in the scheme are as follows:

(1) Install a pump well and drain the clarification pond as described earlier.

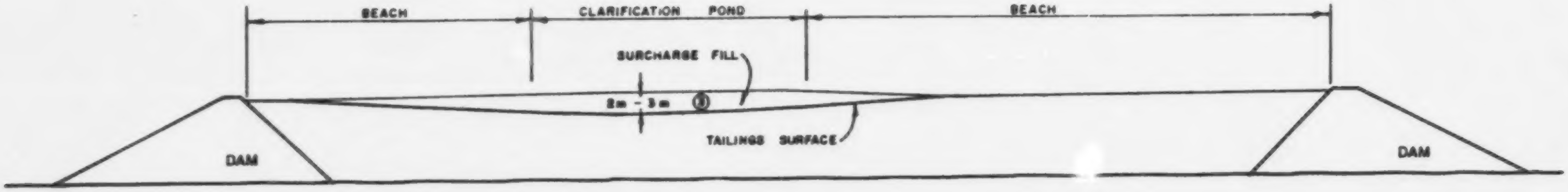
(2) Continue periodic pumping from the well to remove surface runoff for a suitable period until a desiccated crust has formed over the tailings area.

(3) Place a suitable height of surcharge fill in the order of 2 to 3 m, strategically located over the clarificaiton pond area where the greatest settlements are expected to occur based on modelling analyses.

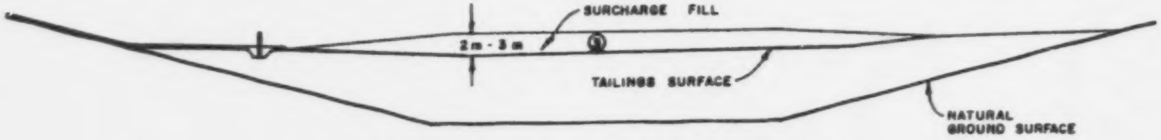

BEACH
CLARIFICATION POND
BEACH
①
TAILINGS SURFACE
DAM
DAM
FOR DETAILS OF PUMP WELL SEE FIG. 8-1
LONGITUDINAL SECTION
ACCESS ROAD TO PUMP WELL (CONSTRUCTED IN WINTER)
①
TAILINGS SURFACE
NATURAL GROUND SURFACE
TRANSVERSE SECTION AT CLARIFICATION POND

SEQUENCE OF OPERATIONS
① INSTALL PUMP WELL AND DRAIN CLARIFICATION POND
② CONTINUE PERIODIC PUMPING TO REMOVE SURFACE RUNOFF AND ALLOW DESICCATION OF TAILINGS TO OCCUR

FIGURE 8-3A
SCHEMATIC SHOWING SEQUENCE OF OPERATION FOR DRAINAGE AND DESICCATION OF TAILINGS


BEACH
CLARIFICATION POND
BEACH
SURCHARGE FILL
2m - 3m
③
TAILINGS SURFACE
DAM
DAM

LONGITUDINAL SECTION


SURCHARGE FILL
2m - 3m
③
TAILINGS SURFACE
NATURAL GROUND SURFACE

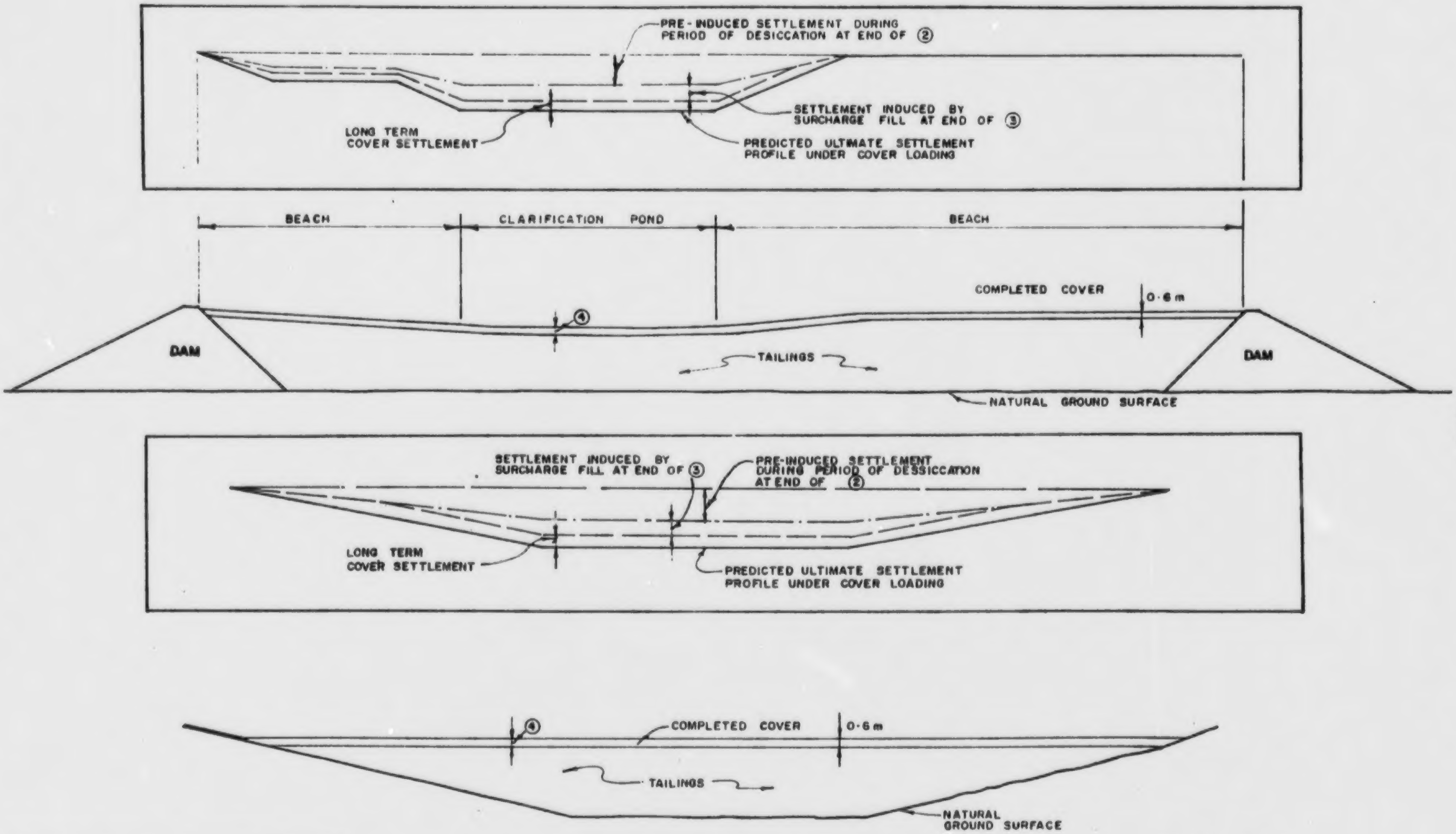
TRANSVERSE SECTION AT CLARIFICATION POND

SEQUENCE OF OPERATION

③ PLACE SURCHARGE FILL STRATEGICALLY LOCATED OVER CLARIFICATION POND AREA, GUIDED BY PREDICTED SETTLEMENT PROFILES SHOWN SCHEMATICALLY IN FIG. 8-3C

FIGURE 8 - 3B

SCHEMATIC SHOWING STRATEGIC PLACEMENT OF SURCHARGE FILL


PRE-INDUCED SETTLEMENT DURING PERIOD OF DESICCATION AT END OF ②
SETTLEMENT INDUCED BY SURCHARGE FILL AT END OF ③
LONG TERM COVER SETTLEMENT
PREDICTED ULTIMATE SETTLEMENT PROFILE UNDER COVER LOADING
BEACH
CLARIFICATION POND
BEACH
COMPLETED COVER
0·6 m
④
DAM
TAILINGS
DAM
NATURAL GROUND SURFACE
SETTLEMENT INDUCED BY SURCHARGE FILL AT END OF ③
PRE-INDUCED SETTLEMENT DURING PERIOD OF DESSICCATION AT END OF ②
LONG TERM COVER SETTLEMENT
PREDICTED ULTIMATE SETTLEMENT PROFILE UNDER COVER LOADING
④
COMPLETED COVER
0·6 m
TAILINGS
NATURAL GROUND SURFACE

SEQUANCE OF OPERATIONS
④ SPREAD SURCHARGE FILL AND COMPLETE COVER CONSTRUCTION
⑤ CONSTRUCT SURFACE DRAINAGE SYSTEM

FIGURE 8 - 3C

SCHAMATIC SHOWING COMPLETED COVER AND PRE - INDUCED AND RESIDUAL SETTLEMENT

(4) Monitor settlement induced by surcharge loading until a sufficient amount has occurred to essentially eliminate differential settlements of the cover in the long term based on predicted settlement profiles shown shcematically in Figure 8-3C.

(5) Spread surcharge fill and complete cover construction over the entire tailings area.

(6) Construct a surface drainage system.

By this method sufficient settlements of the tailings surface are pre-induced before construction of the cover is completed so that only small and relatively uniform settlements of the cover will occur in the long term, thereby ensuring the integrity of the surface drainage system which is constructed after completion of the cover. Depending on site specific conditions, the time frame for such a decommissioining scheme would probably be over a period of 3 to 5 years after the end of tailings deposition, with the greater part of the construction activities carried out in the last 1 to 2 years.

A major advantage of this scheme is that simple proven construction methods are used without the need for special equipment and procedures. The work can be carried out using local labour or available mine crews and earth moving equipment if desired. For projects located in remote areas, this is a particularly attractive consideration.

8.2 Other Methods

A number of technically feasible methods for in-situ dewatering of the tailings are briefly described herein for completeness. These include:

- deep wells
- well-points
- sand drains and wicks
- electro-osmosis

The results of this study suggest that there is relatively little merit for using these methods unless timing for completion of decommissioining is critical. In any event, most of these methods require reasonably good access to the tailings surface and the installations involved are more favourably carried out under non-freezing weather. Therefore, a period of initial drainage and desiccation similar to the scheme described in 8.1 above will stili be required and thus the benefit from these methods in reducing the time for decommissioning would probably be relatively small.

Detailed descriptions of these methods and their design requirements together with comparative costs are given by Wardwell, Nelson, Abt and Staub (1983) for application in uranium tailings in the United States. Technical details required in implementation, degree of operational difficulties involved and costs, are of course highly site specific particularly in the Canadian environment. However, the relative costs given by the authors could be used as a guideline for relative ranking of the methods. A few additional comments related to each method are given below.

8.2.1 Deep Wells

The effectiveness of deep wells in the present application is severely limited by the following factors:

- The cone of influence around a deep well is limited to a very small radius of probably 5 m or less (Wardwell et al, 1983) due to generally low permeability of the tailings thus requiring a large number of wells.

- The conical shape of the lowered water table around each well creates a highly non-uniform surcharging effect over the tailings area, thus resulting in possible undesirable differential settlements of the cover.
- A full depth sand filter surrounding the well casing is required to prevent migration of tailings into the well, making the installation difficult and expensive.
- Formation of a consolidated layer of low permeability tailings around the sand filter is expected to significantly reduce the effectiveness of the well in time.
- An extensive network of piping will be required to convey the water from closely spaced wells over a large area, to a suitable treatment facility.

8.2.2 Well-Points

Unless eductor type well-points are used, the limited depth of water level lowering by suction pump will require even closer spacing than the deep well system described above. Other disadvantages of the deep well system referred to earlier are also applicable in this case.

Use of a vacuum well-point system (Holtz and Wager, 1975) would increase the effectiveness but also the cost of installation and operation.

8.2.3 Sand Drains and Wicks

These systems are only capable of dissipating excess pore-water pressures already present within the tailings and thus are only effective in combination with surcharge loading. In addition they are subjected to similar limitations to the deep well system. Wardwell et al (1983) indicated that experience with this type of installation in phosphate tailings showed generally poor ineffective performance probably due to clogging by fines migrating through the filter materials.

8.2.4 Electro-Osmosis

Electro-osmosis is essentially a special well-point system with accelerated flow of water into the well-points induced by passing an electric current between the well-points acting as cathodes and corresponding rows of anodes of steel or other conductive metal rods driven into the ground. It has been successfully used for stabilizing soft subsoils beneath embankments or pavements (Casagrande, Wade, Wakely and Loughney, 1981; Halton, Loughney and Winter, 1965). Limitation for application in the present instance is mainly from the high costs involved, particularly considering the high salinity of the pore fluid which will result in very large power consumption.

9.0 AREAS REQUIRING FURTHER RESEARCH AND DEVELOPMENT

A number of areas of uncertainty were encountered during the course of this study which prevented definitive modelling of the long term consolidation behaviour of the three typical types of tailings in Canada. In order to improve the state-of-the-art, further work in these areas is required as discussed below.

9.1 Predictive Methodology

Limitations in currently available computer programs suitable for application by the practising engineer have been discussed in Section 4.0. Because of the many inter-related processes which occur simultaneously within a uranium tailings dpeosit, particularly after completion of deposition, an integrated program taking into account all the processes ongoing at any time is required to model the behaviour of the tailings accurately. The program PCDDF (Cargill, 1985) has accomplished a large part of this objective by integrating the interaction of desiccation and consolidation into the analysis. In its present form, however, the program lacks the capability of dealing with

- surcharge loadings on the surface of the consolidating material
- consolidation due to negative pore-water pressures created by gravity drainage as the water level is lowered below the surface of the consolidating material,
- underdrainage through natural formation below the deposited tailings, and
- consolidation of multi-layered systems.

It is believed that modification of the program to account for surcharge effects can be readily accomplished. To deal with the other two problems, however, it is anticipated that a considerable research and development effort would be required.

Methodologies for quantitative prediction of freeze-thaw effects are also lacking at present. Overall modelling methodology of the water balance in an entire tailings deposit such as that given by Ferguson, Hutchison and Schiffman (1985), is also required to predict the rate and extent of water level lowering in the tailings.

9.2 Material Properties

As disscussed in 5.0, a major obstacle encountered in the present study is the lack of information on the properties of uranium tailings related to large strain consolidation, unsaturated consolidation and desiccation processes. A concerted effort is required to produce a body of knowledge required for input into analytical models to enable accurate predictions of long term behaviour to be made. Specifically, the compressibility (e- ' relationship) and permeability (e-k relationship) of various types of uranium tailings under very low stress levels need to be further investigated. Empirical parameters related to the desiccation process (e_{SL}, e_{DL}, maximum depth of desiccation) need to be evaluated for various types of tailings under different environmental conditions. The last two parameters are particularly sensitive to site specific conditions and have to be evaluated in the field by large scale monitoring and measurements.

To enable overall water balance studies to be carried out, accurate detailed information on the hydrogeological conditions surrounding a tailings deposit together with the design details and seepage control characteristics of impoundment structures have to be obtained.

9.3 Field Verification

Regardless of the sophistication of analytical methodologies, it is unrealistic to anticipate that the complex variations of material, hydrologic and environmental conditions in the field be accurately modelled. Field monitoring of consolidation behaviour in terms of time trends of settlements, recession of water level and extent of desiccation, and dissipation of pore-water pressures, will be essential for calibrating or validating the analytical models.

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APPENDIX

STATEMENT OF WORK

CONTINUATION

CONTRACT SERIAL NUMBER / NUMÉRO DE SÉRIE DU CONTRAT

DSS FILE NUMBER – NUMÉRO DU DOSSIER MAS: 15SQ.23241-5-1708

Appendix "A"

STATEMENT OF WORK

Consolidation of Uranium Mill Tailings

Background:

Mineral wastes from mining and milling operations are usually hydraulically transported to a tailings impoundment area as a slurry. Because of process and pumping requirements, the amount of water used in the transport greatly exceeds the free moisture present with the solids at terminal settled density. Decant of the excess water for treatment before release (if necessary) or recycle to process, accentuates the segregation of fine particles from coarse in the tailings impoundment. The process of solids settlement combined with size segregation leads to a poorly consolidated mass with undesireable characteristics.

In the Canadian uranium ore processing industry, tailings have been traditionally deposited as "thin" slurries into impoundment basins with natural processes of sedimentation and segregation allowed to take place. Uranium tailings exhibit enhanced segregation characteristics because of the formation of fine hydrophyllic chemical precipitates in the ore treatment process. Poor consolidation and size segregation enhances hydraulic conductivity and reduces load-bearing capacity, undesireable characteristics for long-term stability and for minimizing the long-term care required.

The National Uranium Tailings Program requires a two part study of the science and practical aspects of uranium tailings consolidation. The first part is a review of the present knowledge of tailings consolidation and will apply this knowledge to the Canadian situation. A list of references to assist with this review is attached. The second part of this study will comprise field and laboratory investigations and analyses of these investigations. It is intended that these investigations

CONTINUATION

CONTRACT SERIAL NUMBER
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15SQ.23241-5-1708

will center on a specific uranium tailings impoundment in Northern Saskatchewan that will be decommissioned in the near future. Access to this specific property will be arranged by the National Uranium Tailings Program. Bidders are requested to not contact the property owner during the bidding process, (Eldor Mines).

It is proposed that the level of effort be split approximately equal between each part of the study.

Part A: Review and Examination of Fundamental Concepts

1. Where information is available, describe the physical and hydraulic characteristics that result from the deposition of uranium tailings in Canada.

2. Describe the fundamental processes involved in consolidation of uranium tailings and the relation of these processes to hydraulic conductivity, compression and load-bearing capacity for saturated and unsaturated tailings.

3. Apply the fundamental processes to the consolidation of existing uranium tailings in Canada. Three types of tailings with their corresponding impoundment system should be considered: Low grade pyritic, medium grade neutral, and high grade arsenic-nickel-clay. Include a discussion of the effects of cold weather.

4. Define the problems resulting from slime-sand segregation, and low consolidation as they relate to long-term environmental aspects of uranium tailings.

5. Describe practical methods to reduce segregation and maximize consolidation during operation of the three different typical impoundment systems. This description should include a discussion of uncertainties, costs and benefits for the long-term.

CONTINUATION

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For purposes of these discussions, it should be assumed that a soil/rip-rap cover of at least 0.6 meter depth will be placed on the tailings after milling has ceased.

6. Describe practical ways to promote consolidation of existing uranium tailings deposits, including the discussion and the assumption in (5) above.

Part B: Test Program and Analyses

1. Make field measurements at, and obtain representative samples of tailings from an active tailings impoundment in Northern Saskatchewan. It is expected that the surface of this impoundment would not support mechanized equipment before mid-January 1986.
2. Conduct laboratory experiments to determine the physical characteristics of the tailings as related to consolidation and segregation.
3. Analyse field and laboratory data and relate to overall impoundment facility.
4. Propose practical methods to consolidate the tailings in situ so that maintenance of a stable cover and minimization of porewater movement is achieved in the short and long-term.

CONTINUATION

CONTRACT SERIAL NUMBER NUMÉRO DE SÉRIE DU CONTRAT

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15SQ.23241-5-1708

References:

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PART

2

NATIONAL
URANIUM
TAILINGS
PROGRAM

Energy, Mines and Resources Canada

Energie, Mines et Ressources Canada

Canada

CONSOLIDATION OF URANIUM MILL TAILINGS

FINAL REPORT - PART B

DSS File No. 15 SQ.23241-5-1708

CONTRACT REPORT NO. OSQ 85-00218

MICROMEDIA

200pp

February 1987

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PART B - FIELD INVESTIGATION

SUMMARY

This report contains the results of a field and laboratory investigation carried out at the Rabbit Lake tailings deposit in northern Saskatchewan and mathematical modelling of the consolidation/desiccation behaviour in relation to cover construction aspects in decommissioning the facility. The work was carried out as the second part of a two-part study commissioned by the National Uranium Tailings Program. In the first part (Part A) of the study, a general review of the current state of the knowledge related to the consolidation of uranium tailings in Canada led to the identification of parameters and development of methodologies for simulating the large strain consolidation/desiccation behaviour of such deposits. Also in Part A of the study, an appraisal of various currently available techniques for potential application in promoting consolidation of tailings deposits was carried out and led to the conclusion that utilization of desiccation by evaporation appeared to be, comparatively speaking, a cost effective and technically simple method in minimizing long term cover settlements. The results of the study in Part A provided direction for the investigation and analyses conducted in Part B as described in this report.

The Rabbit Lake tailings deposit investigated in this part of the study was produced from the milling of medium-grade non-pyrite ore using an acid leach process. The tailings was neutralized by lime treatment prior to co-disposal with the neutralized raffinate stream. About 5.9 million tonnes of tailings were deposited over a 10 year period from 1975 to 1985, in an impoundment area formed by two dams constructed across a broad saddle valley. Conventional open-end discharge was used to deposit the tailings slurry in the impoundment area except for short periods in the early part of the operation when a mobile cyclone unit was used for extracting the coarse fraction of the tailings for dam construction.

Examination of the history and method of deposition of the tailings in the impoundment area led to the identification of three main zones with anticipated major differences in the characteristics of the deposited tailings. Consequently, the field investigation program consisted of a borehole put down through the full depth of the tailings in each of the three zones to obtain samples for laboratory testing. In-situ permeability and shear strength testing was also carried out to supplement the laboratory testing program. Mineralogical analyses on a limited number of samples were also carried out.

The results of the field and laboratory investigation verified the nature of the three main zones within the tailings deposit:

- the clarification pond area with concentration of silty tailings slimes in unfrozen state

- the south beach area with an upper part of predominantly sandy tailings over a lower part of predominantly silty tailings slimes, in essentially unfrozen state with minor frozen layers near the top, and

- the north beach area of predominantly frozen tailings with occasional thin layers of clear ice.

The mineralogical analyses revealed the presence of trichlorite, gypsum and bassanite in the tailings. Scanning electron microscope work characterized the particle shapes and sizes of the tailings.

Information on the properties of the unfrozen tailings obtained in the investigation enabled one-dimensional large strain consolidation/desiccation models to be analysed. For discussion purposes in this report, frozen zones were assumed to either remain permanently frozen or become completely thawed some time in the future. Settlement profiles along longitudinal and transverse sections of the tailings deposit were developed under these scenarios. These predicted settlement profiles indicated that, if the frozen tailings remains frozen in the long term, a strategy of deferred cover construction would be effective in reducing differential settlements of the cover to acceptable levels. The method as described in the report is necessarily conceptual and schematic in nature. Its success is dependent on the effective drainage of surface water to enable evaporation from the tailings surface to take place. A number of basic methodologies to enhance surface drainage are described in the report. A definitive design of the surface drainage measures is however, beyond the scope of this study.

In the event of the frozen tailings thawing out in the future, it is anticipated large settlements could occur although at very slow rates and eventually, reversal of surface gradients and disruption of surface drainage facility could result. As discussed in the report, such an event is extremely unlikely at the project site due to its northern location in an area of known occurrence of sporadic permafrost.

GEOCON INC.

W.F. Tao

W.F. Tao
Project Manager

February 1987

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APPENDIX II - PIEZOCONE TEST RESULTS AND INTERPRETATION

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APPENDIX IV - STATEMENT OF WORK

1.0 INTRODUCTION

This report presents the second part (Part B) of a two-part study on the consolidation behaviour of uranium mill tailings in Canada, which was commissioned by the National Uranium Tailings Program under Supply and Services Canada Contract No. 15SQ.23241-5-1708. The stipulated statement of work is included in Appendix IV of this report. Specifically, this part of the study includes a field and laboratory investigation of the Rabbit Lake tailings deposit located in northern Saskatchewan and the application of a mathematical model to simulate its consolidation/desiccation behaviour in connection with a preliminary examination of the potential impact of long term settlements on the planning of decommissioning strategy for the facility.

Deposition of a medium-grade non-pyritic tailings in the impoundment area took place over a ten year period from 1975 to 1985 and resulted in an accumulated thickness of some 20 m of tailings. Decommissioning of the facility is planned in the near future and therefore it presents itself as an ideal case for the study at hand.

A detailed account of the site conditions and the history of operation of the facility is given in Section 2.0.

Information from an investigation carried out in 1981 (Gulf Minerals, 1981) enabled a preliminary examination of the problem in the first part (Part A) of the study and provided insight into the areas of deficiency in information for sufficiently definitive analyses of the consolidation/desiccation behaviour of the tailings deposit. Guided by that preliminary study and within the time frame and level of funding for the project, a sampling and testing program was carried out in this part of the study to supplement the earlier investigation. The scope, procedures and results of the investigation are given in Section 3.0 of this report.

The influence of the decommissioning schedule and strategy on long term settlement is discussed in Section 4.0.

Predictive analyses on self-weight consolidation, desiccation and settlement under cover are presented in Section 5.0.

2.0 THE RABBIT LAKE TAILINGS DEPOSIT

The Rabbit Lake Mine tailings deposit was described briefly in the report on Part A of this study. A more detailed description of the relevant site conditions and the history of the operation is presented below to assist in refining the modelling of the consolidation/desiccation behaviour of the tailings deposit.

2.1 Site Conditions

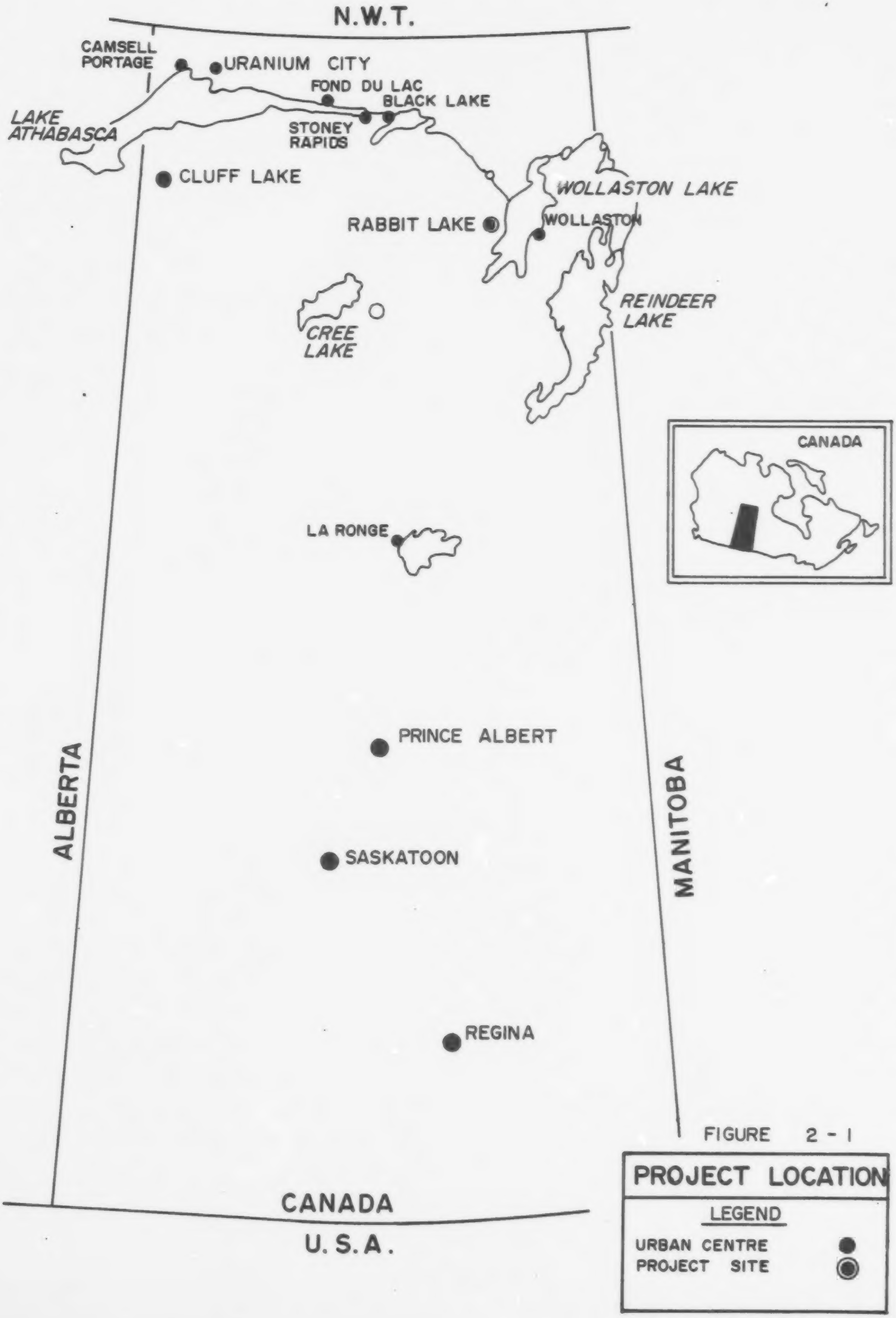
2.1.1 Location

The Rabbit Lake Mine is located about 750 km north of Saskatoon, Saskatchewan as shown in Figure 2-1.

2.1.2 Meteorology

The site is in an area of sporadic permafrost with short, warm summers and long cold winters with about 5500 degree days below freezing.

Annual average precipitation is about 546 mm with 403 mm in the form of rainfall. Average monthly precipitation and rainfall, recorded at the Collins Bay weather station in the immediate vicinity of the site between 1972 and 1984 are shown in Table 2-1.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CAMSELL PORTAGE
URANIUM CITY
FOND DU LAC
BLACK LAKE
LAKE ATHABASCA
STONEY RAPIDS
CLUFF LAKE
WOLLASTON LAKE
RABBIT LAKE
WOLLASTON
REINDEER LAKE
CREE LAKE
CANADA
LA RONGE
PRINCE ALBERT
ALBERTA
MANITOBA
SASKATOON
REGINA
CANADA
U.S.A.
LEGEND
URBAN CENTRE ●
PROJECT SITE ◉

FIGURE 2 - 1

PROJECT LOCATION

(GULF MINERALS, 1980a)

TABLE 2-1 RECORDED AVERAGE MONTHLY PRECIPITATION AND RAINFALL COLLINS BAY STATION (1972-1984)

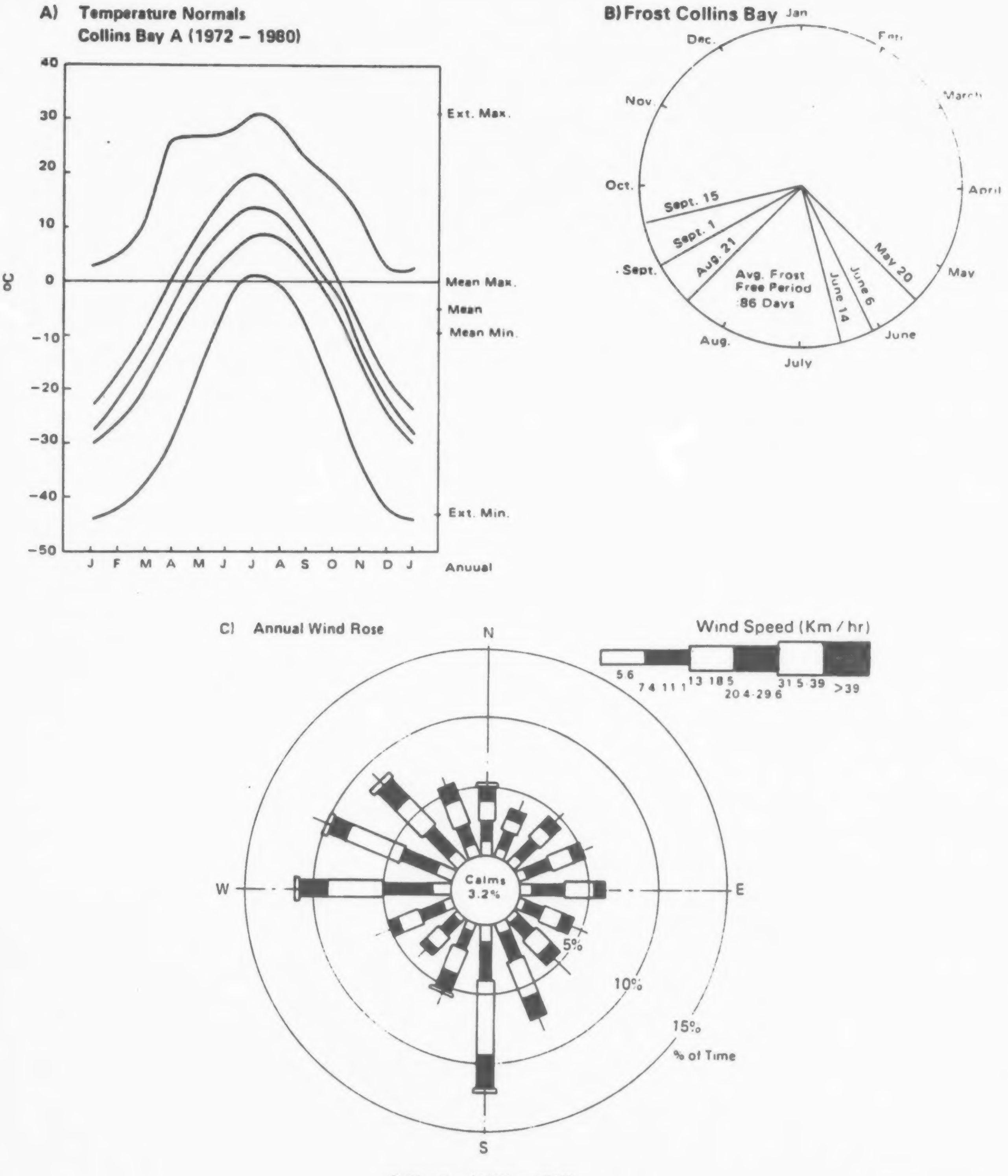
Month	Average Precipitation - mm	Average Rainfall - mm
January	19.1	0.0
February	15.5	0.3
March	24.9	0.0
April	25.1	5.1
May	43.2	34.8
June	70.1	63.2
July	108.5	108.5
August	72.4	71.9
September	63.5	53.6
October	44.2	14.2
November	26.9	1.5
December	24.9	0.0

No evaporation records are kept at the Collins Bay weather station. From the Hydrological Atlas of Canada, an average annual evaporation rate of 514 mm was estimated. By pro-rating to the average monthly evaporation rates recorded at the nearest station at Cree Lake, the estimated average monthly evaporation rates at the site are shown in Table 2-2.

TABLE 2-2 ESTIMATED AVERAGE MONTHLY EVAPORATION AT RABBIT LAKE

Month	Average Evaporation - mm
January - April	0
May	118
June	119
July	128
August	98
September	51
October - December	0

Processed information on temperature, frost and wind recorded at Collins Bay between 1972 and 1980 are shown in Figure 2-2.



Collins Bay (1972 – 1980)

FIGURE 2-2 CLIMATIC NORMALS COLLINS BAY
(MacLAREN - GEOCON, 1984)

2.1.3 Surface Conditions

The Rabbit Lake tailings area is located about two miles south of the Mine/Mill complex. Two earth dams, designated the North Dam and the South Dam, were constructed across an elongated valley between two north-south trending ridges immediately to the west of Parks Lake for impoundment of tailings (Figure 2-3). A pump-barge located on the west side of the Tailings Area decants clarified water via a pipeline and treatment facilities into a Precipitation Pond downstream of the South Dam. Treated water from the Precipitation Pond is discharged into a poorly defined creek trending south through low-lying swampy ground into Hidden Bay. A diversion channel along the west side of the Tailings Pond intercepts surface storm run-off from a relatively small watershed, some 40 hectares in area, and discharges it directly into the creek below the Precipitation Pond. The ridge along the east side of the Tailings Area is narrow and steep-sided, with a very small catchment area draining into the Tailings Pond. No diversion channel is present on this side.

2.1.4 Subsurface Conditions

The general area of the project site is known to have a shallow groundwater table generally following the topographic features with swampy ground in low-lying areas and natural springs near the foot of ridges. The overall trend of groundwater flow is easterly from the high ground forming the watershed between the Rabbit Lake Basin and the Parks Lake Basin, towards Parks Lake which is a local groundwater discharge area.

The overburden in the general area of the site consists of varying thicknessess of ablation and basal glacial till drift (Drake, 1971). Glacial striae indicate that the glacial ice sheet advanced from north-east to

Geocon

○ **Borehole in Tailings 1981**
(GULF MINERALS, 1981b)

NOTES

1 FOR SECTIONS SEE FIGURES 2-4 AND 2-5.

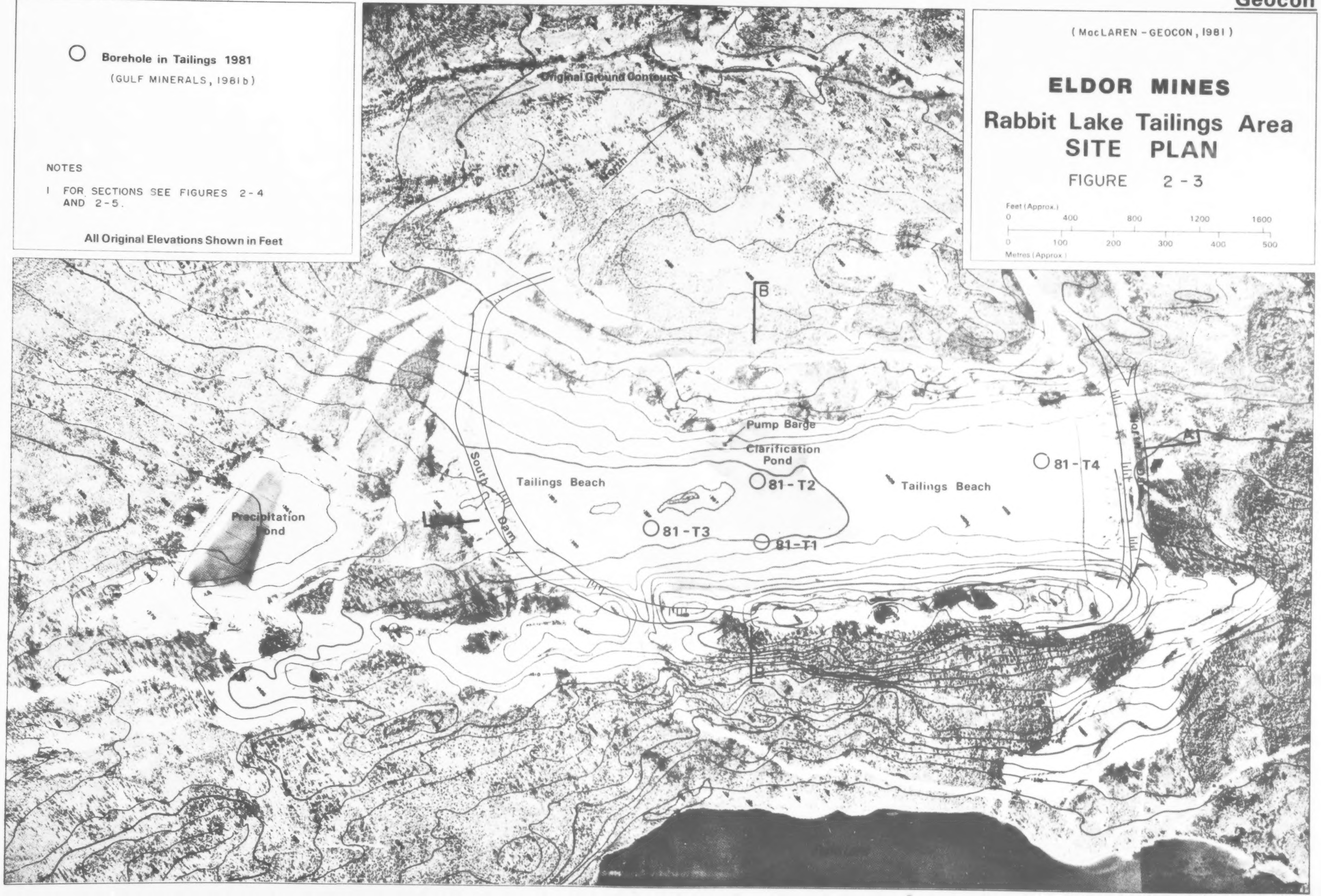
All Original Elevations Shown in Feet

(MacLAREN - GEOCON, 1981)

ELDOR MINES

Rabbit Lake Tailings Area SITE PLAN

FIGURE 2-3


Feet (Approx.)
0
400
800
1200
1600
0
100
200
300
400
500
Metres (Approx.)
Original Ground Contours
North
B
A
Pump Barge
Clarification Pond
Tailings Beach
Tailings Beach
South Dam
North Dam
Precipitation Pond
81 - T1
81 - T2
81 - T3
81 - T4
B

south-west and consequently most of the prominent headlands and well exposed shorelines face north-east. Because of the generally thin drift to the east of Rabbit Lake, most glacial deposits are inferred to lie in the lee, to the south-west of the major cliffs or other bedrock outcrops.

In the vicinity of the tailings area, a veneer of glacial till generally occurs directly overlying bedrock, with a thin muskeg cover over the till in low-lying areas (Wallis, 1971; Geocon, 1972).

The basal till consists of a silty to sandy till with permeability ranging between 1×10^{-5} and 3×10^{-3} cm/sec and the ablation till consists of a gravelly-cobbly material with permeability generally in the 10^{-2} cm/sec order of magnitude (Gulf Minerals 1980b).

Bedrock in the general area consists of a complex assemblage of gneisses, quartzite, pegmatite and amphibolite, with secondary permeability ranging over four orders of magnitude from 10^{-7} to 10^{-3} cm/sec. (Geocon 1982).

2.2 History of Dam Construction and Tailings Deposition

The impoundment dams were constructed of zoned, compacted earth fill using local glacial till borrow material for the bulk of the construction (Geocon 1973, 1978, 1979, 1980, 1981, 1982 and 1983). Select low permeability silty till with a minimum of 20 percent minus No. 200 mesh size fraction was used in the upstream portions of the dams to minimize seepage through them. Coarser sandy till borrow was used in the downstream zones to provide strength and stability. Initially the coarse fraction of the tailings was also extracted by using a mobile cyclone unit for constructing part of the downstream zone of the South Dam. This practice was terminated in 1980 and the balance of the South Dam completed using till borrow material. A system of internal drains constructed of crushed rock and coarse sand and gravel borrow material was incorporated in each dam to relieve pore water pressures within it for better structural stability. A compacted, low permeability till filled cut-off trench was used to reduce seepage through the ablation till stratum below each dam.

The starter dams were constructed in 1974 and deposition of tailings into the area began in late 1975. Raising of the dams was carried out annually from 1977 to 1983 to maintain a minimum freeboard of 1.5 m, as indicated in Table 2-3.

TABLE 2-3 HISTORY OF DAM CONSTRUCTION

YEAR	CREST ELEVATION - M	
	NORTH DAM	SOUTH DAM
1974 (Starter Dams)	433.8	433.5
1977	437.5	436.6
1978	440.5	439.0
1979	440.8	440.8
1980	443.0	443.0
1981	445.1	445.1
1982	447.4	447.4
1983	449.7	449.7

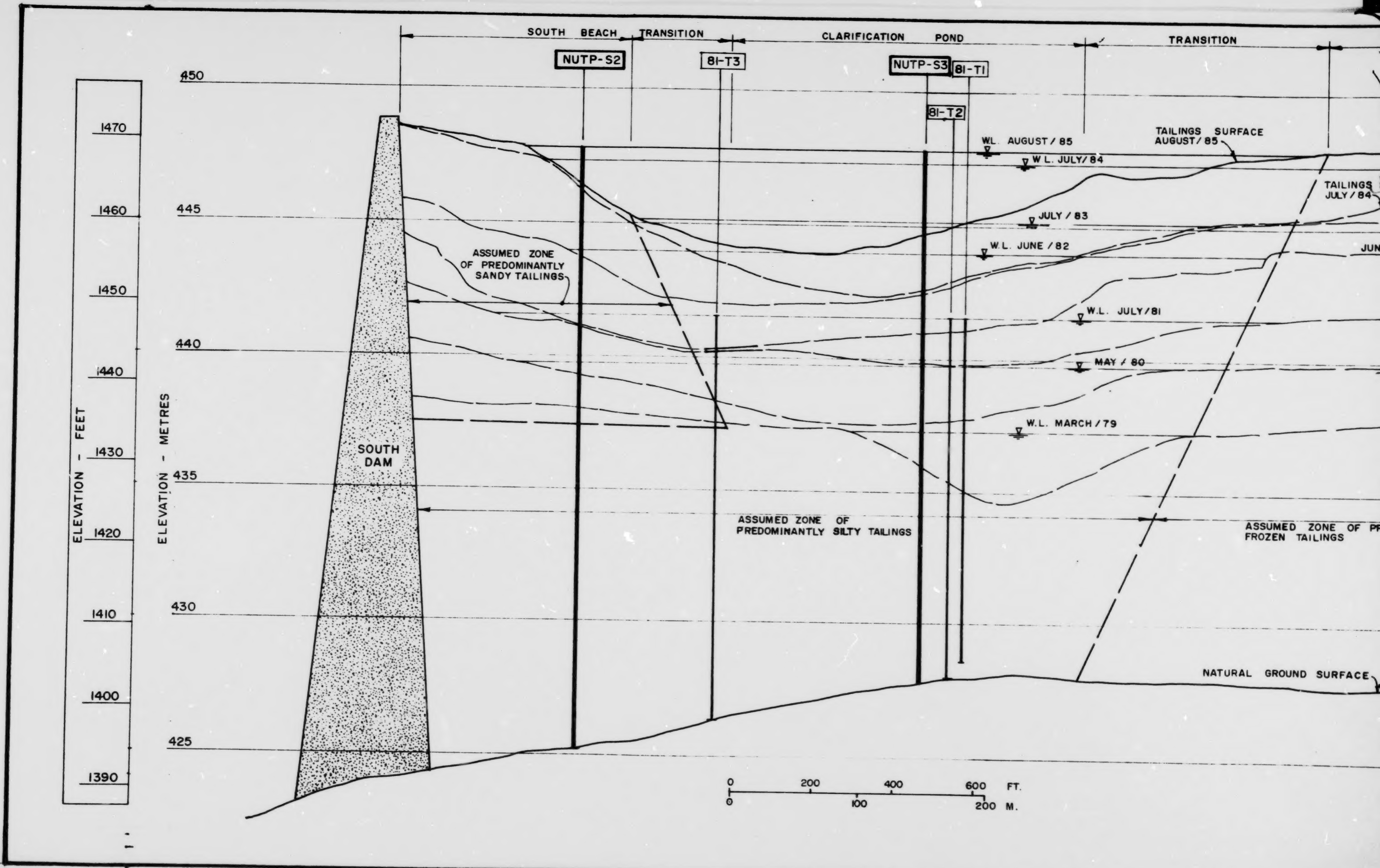
Milling practice is described in Gulf Minerals (1980a). The design milling rate was 1814 tonnes (2000 tons) per day on an average ore grade of 0.37 percent U_3O_8. The ore was ground by a wet process to 100 percent minus No. 28 mesh sizes with an average of 50 to 55 percent minus No. 200 mesh sizes and the uranium extracted by an acid leach process, using sulphuric acid, at a pH of 1.4 to 1.5. After removal of the dissolved uranium by counter current decantation through a series of six washing thickeners, the waste tailings was discharged as an underflow slurry of 55 percent solids and combined with the raffinate bleed stream in a neutralizing tank where slaked lime was added at the rate of 30 kg per tonne (60 pounds per ton) of ore, raising the slurry to a pH of 8 before discharging via a 250 mm polyethylene pipeline to the tailings impoundment area. A total of 5.9 million tonnes (6.5 million tons) of tailings were actually deposited into the impoundment area according to the milling record shown in Table 2-4.

TABLE 2-4 MILLING RATE RECORD

YEAR	MILL THROUGHPUT - TONNES
1975	130,000
1976	397,000
1977	547,000
1978	573,000
1979	650,000
1980	666,000
1981	677,000
1982	616,000
1983	620,000
1984	642,000
1985	404,000
	5,922,000

The predominant point of discharge of the tailings slurry was located on natural high ground on the north-west of the area near the west end of the North Dam. Winter discharge in particular, was almost entirely from this point. Prior to 1980, intermittent operation of a mobile cyclone on the South Dam to extract the coarse fraction of the tailing for dam construction resulted in deposition of the fine fraction of silty tailings (slimes) immediately upstream of the South Dam. End discharge from points located on the South Dam were periodically carried out since 1979, to form a beach against the dam. Periodical end discharge from various points on the east side of the tailings impoundment area was also carried out since 1981.

The manner of deposition described above combined with decantation of clarified water by a pump barge located about mid-way between the North Dam and the South Dam (Figure 2-3), resulted in a clarification pond occupying essentially the southern half of the area with free water against the South Dam most of the time during the initial period up to 1979. Since then, displacement of the southern limit of the clarification pond resulted in its central location as shown in Figure 2-3. The tailings surface profile and clarification pond water levels at various times are shown in Figures 2-4 and 2-5.


SOUTH BEACH
TRANSITION
CLARIFICATION POND
TRANSITION
NUTP-S2
81-T3
NUTP-S3
81-T1
81-T2
ELEVATION - FEET
1470
1460
1450
1440
1430
1420
1410
1400
1390
ELEVATION - METRES
450
445
440
435
430
425
W.L. AUGUST / 85
W.L. JULY/84
TAILINGS SURFACE AUGUST/ 85
TAILINGS JULY / 84
JULY / 83
W.L. JUNE / 82
JUN
W.L. JULY/81
MAY / 80
W.L. MARCH / 79
ASSUMED ZONE OF PREDOMINANTLY SANDY TAILINGS
SOUTH DAM
ASSUMED ZONE OF PREDOMINANTLY SILTY TAILINGS
ASSUMED ZONE OF PR
FROZEN TAILINGS
NATURAL GROUND SURFACE
0
200
400
600
FT.
0
100
200
M.

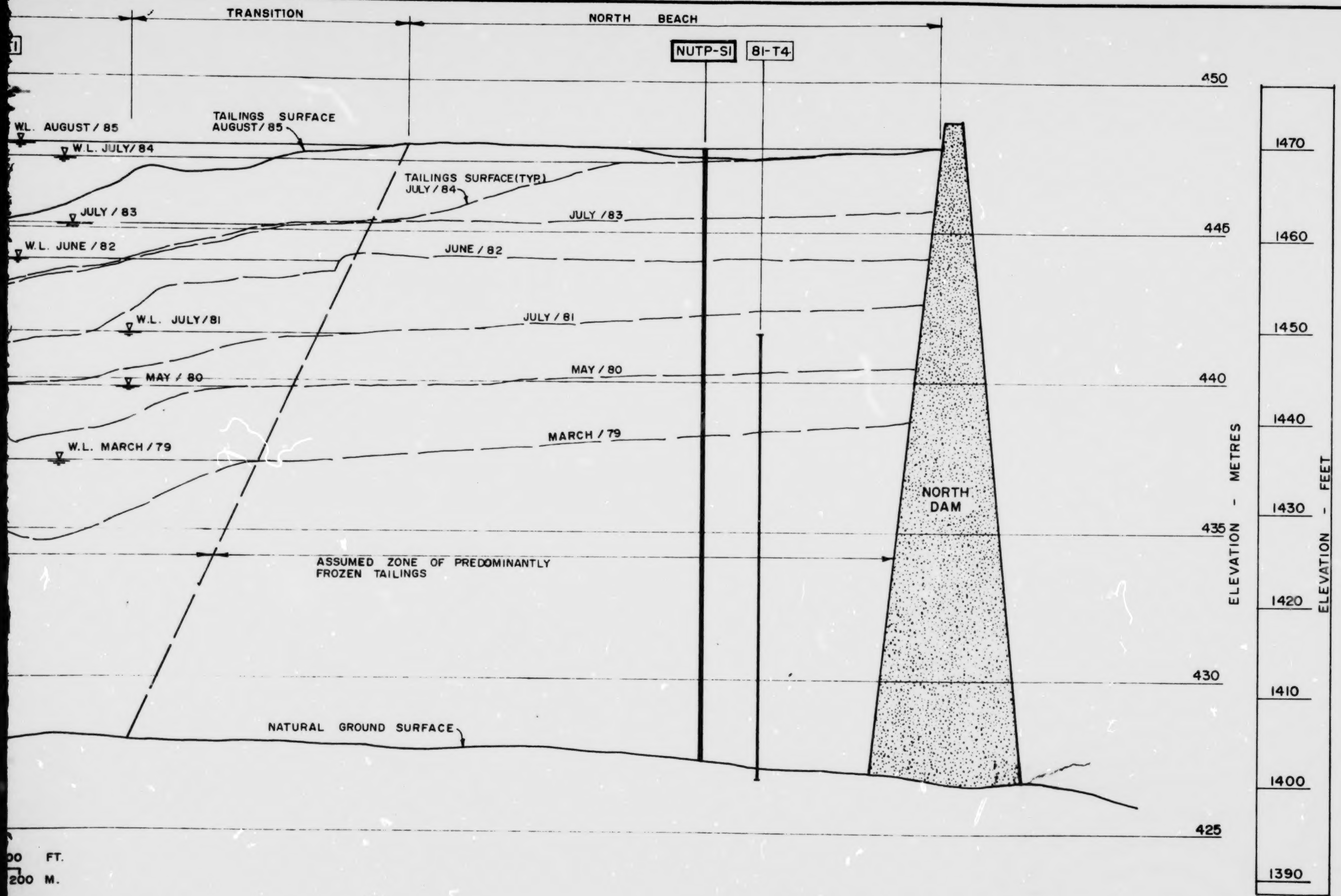

TRANSITION
NORTH BEACH
NUTP-SI
81-T4
450
W.L. AUGUST / 85
TAILINGS SURFACE AUGUST / 85
W.L. JULY / 84
TAILINGS SURFACE(TYP.) JULY / 84
JULY / 83
JULY / 83
445
W.L. JUNE / 82
JUNE / 82
W.L. JULY/81
JULY / 81
MAY / 80
MAY / 80
440
W.L. MARCH / 79
MARCH / 79
NORTH DAM
435
ASSUMED ZONE OF PREDOMINANTLY FROZEN TAILINGS
430
NATURAL GROUND SURFACE
425
ELEVATION - METRES
1470
1460
1450
1440
1430
1420
1410
1400
1390
ELEVATION - FEET
00 FT.
200 M.

FIGURE 2 - 4
RABBIT LAKE TAILINGS AREA
SECTION A- A (SEE FIG. 2-3)

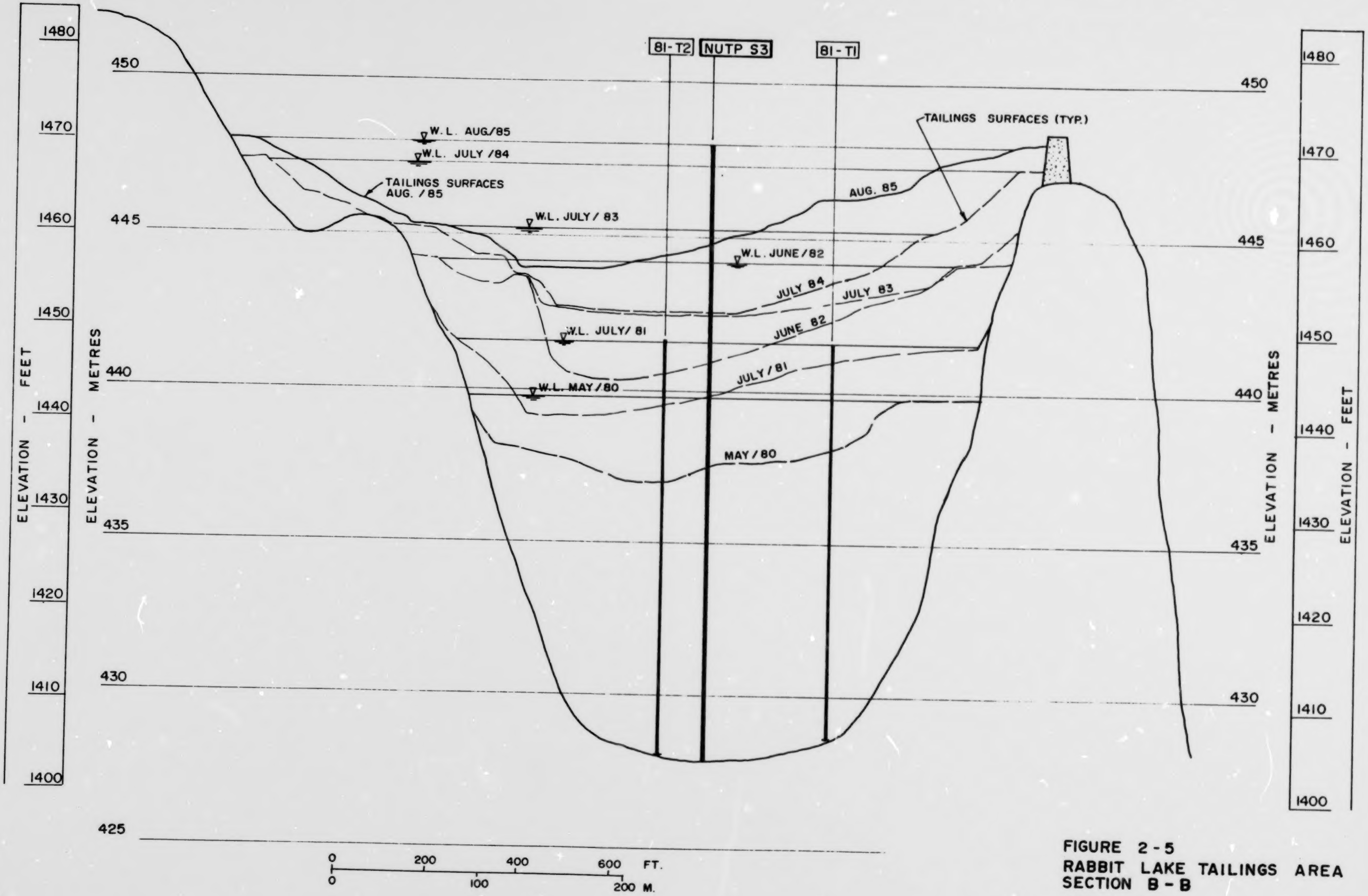

81-T2
NUTP S3
81-TI
TAILINGS SURFACES (TYP.)
W.L. AUG/85
W.L. JULY/84
TAILINGS SURFACES AUG./85
W.L. JULY/83
AUG. 85
W.L. JUNE/82
JULY 84
JULY 83
JUNE 82
W.L. JULY/81
JULY/81
W.L. MAY/80
MAY/80
ELEVATION - FEET
ELEVATION - METRES
1480
1470
1460
1450
1440
1430
1420
1410
1400
450
445
440
435
430
425
0
200
400
600
FT.
0
100
200
M.

FIGURE 2-5
RABBIT LAKE TAILINGS AREA
SECTION B-B
(SEE FIG. 2-3)

3.0 GEOTECHNICAL INVESTIGATION

3.1 Purpose and Scope

In the first part of this study, the geotechnical properties of uranium tailings required in theoretical modelling of large strain consolidation and desiccation behaviour of such deposits were identified. In the case of the Rabbit Lake tailings deposit selected for site specific study, existing information given in Gulf Minerals (1981b) enabled a preliminary series of analyses to be carried out. The results of those analyses illustrated the need for additional information on the compressibility and permeability characteristics of the tailings under very low stress levels and on the limit void ratios (saturation limit, e_{sL} and desiccation limit, e_{DL}) related to the desiccation behaviour of the tailings. The main objective of the geotechnical investigation in the present study was therefore focussed on the determination of these material properties by field and laboratory testing techniques.

The scope of the geotechnical investigation was confined to characterization of the tailings within typical areas to enable one-dimensional models representative of each area to be analysed. On this premise, three major areas anticipated to have distinctly different characteristics were identified for investigation. These are:

- the clarification pond area,
- the south beach area, and
- the north beach area

Variations in properties and thickness of the tailings within each area are of course to be expected. Investigation of such variations in detail is, however, beyond the scope of this study.

In accordance with the objective and scope as outlined above, the program of investigation includes:

- a borehole put down within each area to obtain suitable samples for laboratory testing,
- in-situ falling head permeability testing in unfrozen tailings in a borehole adjacent to the sampling borehole,
- in-situ static cone (piezocone) testing to obtain penetration resistance profiles and pore-water pressure dissipation characteristics,
- in-situ vane shear strength tests in unfrozen tailings adjacent to the sampling borehole, using a self-boring Nilcon vane,
- installation of open-well type piezometers to determine pore-water pressure distribution within the tailings,
- laboratory testing to determine void ratio, compressibility and permeability characteristics related to the sedimentation - consolidation process, limit void ratios related to the desiccation process, grain size distribution characteristics and strength - moisture content - degree of saturation relationships, and
- simple mineralogy.

The procedures used and results obtained from the investigation are described in the following subsections.

3.2 Method of Investigation

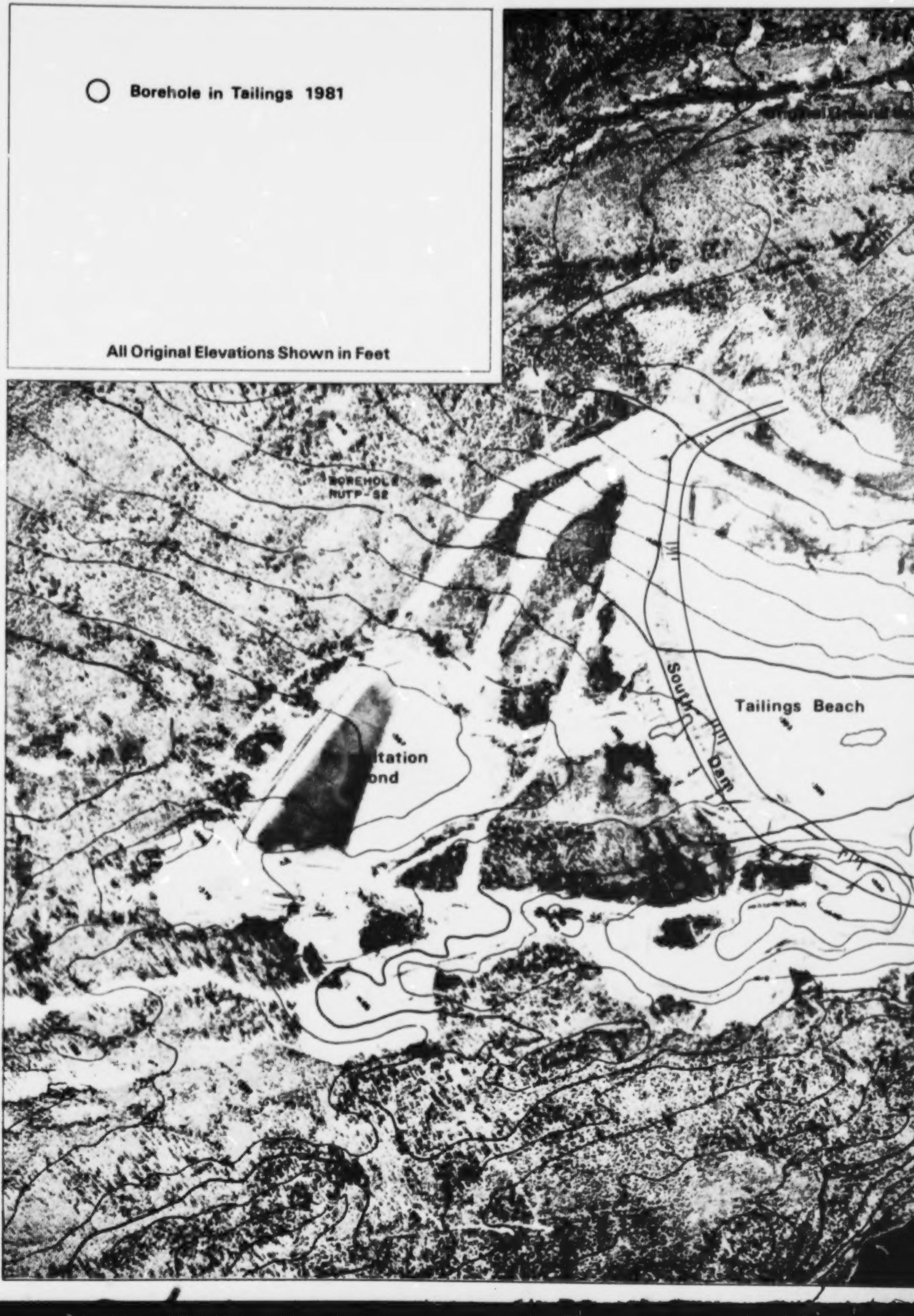
3.2.1 Drilling and Sampling

The drilling and sampling program for Part B of the study was carried out between February 3 and 16, 1986. During this period three boreholes, numbered NUTP-S1 to S3 were put down using a skid mounted Longyear 38 diamond drill. The drill was moved to the drill site and between boreholes by a skidder, which was also employed to open roads within the tailings area for the water truck and personnel vehicles.

Survey stakes marking the intended locations of the boreholes were set out in the field by Eldor Mines from predetermined coordinates provided by Geocon. The actual locations of the boreholes are shown on Figure 3-1. Boreholes NUTP-S1 to S3 were all located on ice surface.

Borehole NUTP-S1 was put down using NW sized casing. Samples of the predominantly frozen tailings were obtained using a triple tube NQ3 core barrel. The drilling fluid was mill water saturated with a liberal amount of salt added to lower its freezing point. It was chilled to slightly below 0°C by the addition of snow and ice to prevent melting of frozen core during drilling. Upon recovery, the core was immediately examined visually and photographed. All water used for drilling was obtained from the mill and trucked to the drill site.

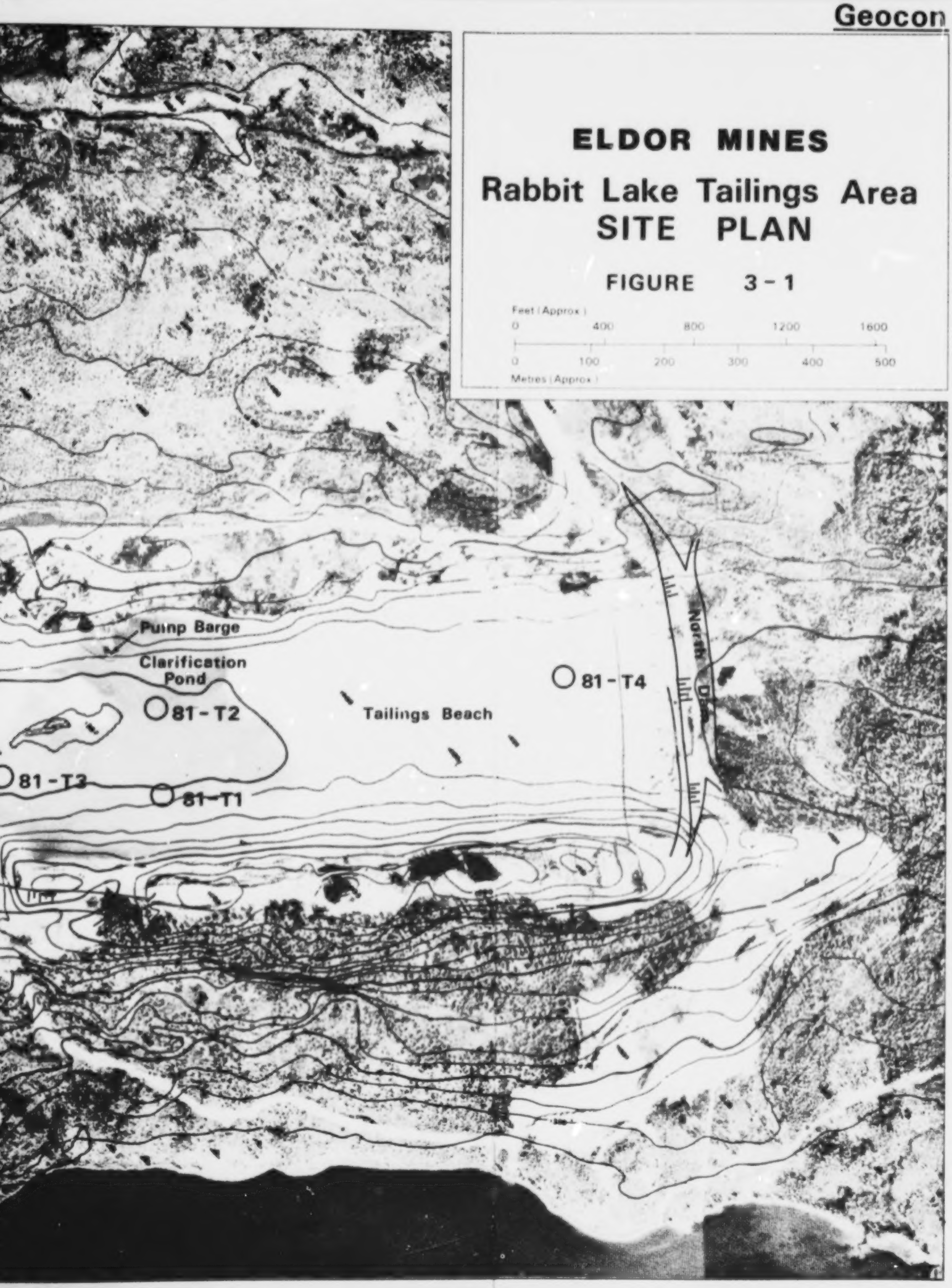
Borehole NUTP-S2 was put down initially using core drilling techniques as for Borehole NUTP-S1 plus standard wash boring procedures. Coring was carried out to about 9 m depth at which depth this technique was discontinued as it became apparent that the tailings at this location was almost completely unfrozen. HW casing was then inserted in the borehole to enable sampling using a 76 mm diameter Osterberg piston sampler to be carried out at 1.5 m intervals.


BOREHOLE
NUTP-S2
tation
ond
South Dam
Tailings Beach

○ Borehole in Tailings 1981

All Original Elevations Shown in Feet

Geocon



ELDOR MINES

Rabbit Lake Tailings Area SITE PLAN

FIGURE 3 - 1

Feet (Approx.)
0
400
800
1200
1600
0
100
200
300
400
500
Metres (Approx.)
Pump Barge
Clarification Pond
81 - T2
81 - T3
81 - T1
Tailings Beach
81 - T4
North Dam

In order to enhance recovery, the crimp at the lower end of the standard sampling tubes was filed away. As well, the grease coating the inside of the tube was burned off and the inside of the tube was cleaned before being used for sample recovery. Upon recovery, the tubes were sealed in plastic bags, wrapped in about 100 mm of fiberglass insulation and carried to a heated truck waiting on the South Dam, which then transported the samples to the field laboratory. These precautions minimized disturbance during sampling and transport.

Borehole NUTP-S3 was put down using the Osterberg sampler in essentially the same manner as Borehole NUTP-S2. However, samples were obtained continuously instead of at 1.5 m intervals. Poor recovery of samples was experienced to a depth of about 10.5 m. As discussed later in Subsection 3.2.6, the tailings to this depth was very loose and of low strength so that sufficient adhesion could not be developed to retain the sample inside the tube even after allowing the sampler to remain undisturbed for 30 minutes before retrieving it from the borehole.

Detailed records of the boreholes are presented in Appendix I.

3.2.2 Piezometer Installation

Upon completion of each borehole, a nest of three Casagrande type piezometers were installed so as to permit observation of the piezometric levels. At each of the locations of Boreholes NUTP-S2 and -S3 an additional hole was also put down to enable another nest of piezometers, also of the Casagrande type, to be installed. Details of the piezometer installations are shown in Table 3-1. Locations of the piezometer tips in elevation are also shown graphically on the individual Boring Logs in Appendix I.

TABLE 3-1

INSTALLATION DETAILS
OPEN-WELL TYPE PIEZOMETERS

Location	Piezometer	Date Installed	Tip Elev (m)	Tip Depth (m)	
				Below Pond W.L.	Below Tailings Surface
NUTP-S1	1	Feb 6, 1986	427.4	21.1	20.9
	2	"	436.4	12.2	12.0
	3	"	440.9	7.6	7.4
NUTP-S2	1	Feb 16, 1986	425.6	22.9	20.8
	2	"	428.7	19.8	17.7
	3	"	431.7	16.8	14.7
	4	"	436.3	12.2	10.1
	5	"	440.9	7.6	5.5
	6	"	443.9	4.6	2.5
NUTP-S3	1	Feb 12, 1986	428.7	19.8	15.8
	2	"	431.8	16.8	12.8
	3	"	436.4	12.2	8.2
	4	"	441.0	7.6	3.6
	5	"	444.0	4.6	0.6

In an attempt to avoid freezing of the water within the piezometer standpipes a special antifreeze, with a specific gravity of 1.0, was added to the piezometer standpipes immediately after installation. However due to dilution of the antifreeze by the water already present within the tubes, this measure met with only limited success as a number of piezometers were frozen within one day after installation. Observed piezometric levels are contained in Section 3.3.

3.2.3 Progress of Sedimentation

Adjacent to Borehole NUTP-S3, the state of sedimentation of the tailings slurry was examined by determining the density profile of the fluid above the settled tailings. For this purpose a special hydrometer was fabricated from PVC pipe and fittings. The hydrometer bulb has a displacement of about 4 litres. The stem of the hydrometer was made up of 3.0 and 1.5 m lengths of 26 mm diameter piezometer tubing. The test was conducted by adding a known weight of sand to the bulb, and measuring the depth at which it came to rest under free floating condition. By repeating the test with the addition of small increments of sand ballast loading, the fluid density of the tailings suspension at various depths was determined.

To supplement the observations from the in situ hydrometer tests adjacent to Borehole NUTP-S3, water/slurry samples were obtained from various depths within the clarification pond using a water sampler specially made for this purpose. The sampler consists of a plexiglass cylinder with closed ends and an entry port fitted with a one way valve which can be opened with a line leading from the sampler to the operator at the surface. The volume of the sample collected was about 1 litre and the amount of suspended solids in the water was determined in the laboratory by oven drying.

3.2.4 Piezocone Testing

Adjacent to Boreholes NUTP-S2 and -S3 static cone penetration tests were carried out using a commercial piezocone unit supplied and operated by Foundex Exploration of Vancouver. It is similar in detail to that developed at the University of British Columbia (Campanella, Robertson and Gillespie, 1982) with a tip area of 10 cm^2. A sketch of the piezocone is presented in Figure 3-2. A multi-channel readout/recording unit enabled

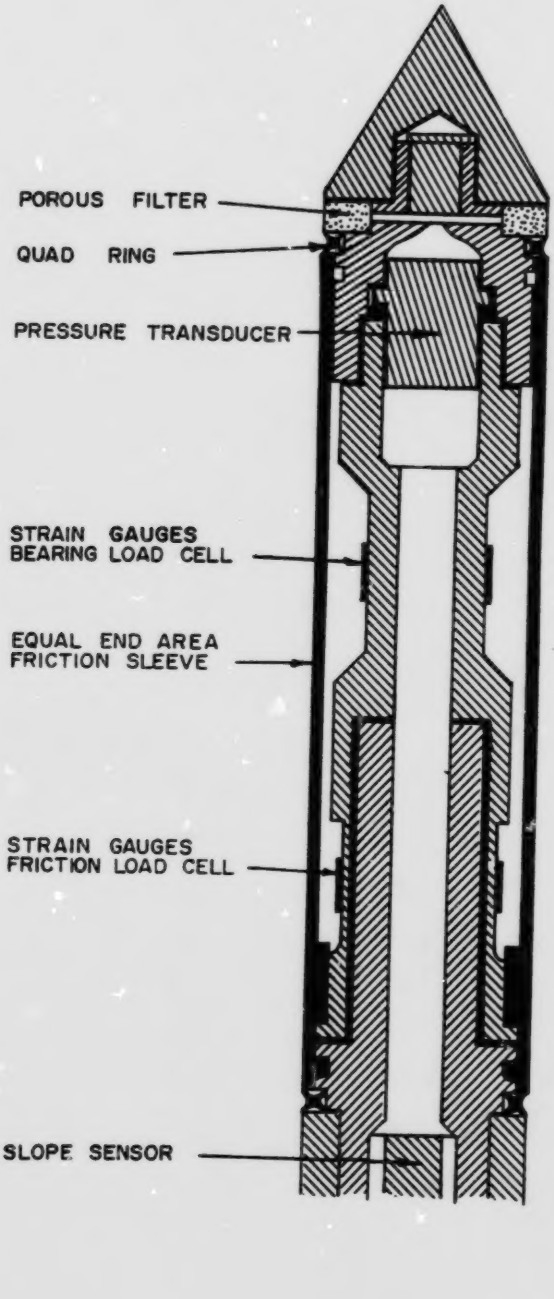

POROUS FILTER
QUAD RING
PRESSURE TRANSDUCER
STRAIN GAUGES BEARING LOAD CELL
EQUAL END AREA FRICTION SLEEVE
STRAIN GAUGES FRICTION LOAD CELL
SLOPE SENSOR

FIGURE 3 - 2 SCHEMATIC OF PIEZOCONE TIP
(After CAMPANELLA, ROBERTSON AND GILLESPIE 1982)

end resistance, pore water pressure, side friction and differential pore water pressure to be monitored continuously during the tests. From this information, undrained shear strength, friction ratio and relative density were deduced. Dissipation tests were also carried out at 2 m depth intervals from which coefficient of consolidation and permeability values were computed. Methodologies for interpretation of the piezocone test data are described in detail by Robertson and Campanella, 1983.

The hydraulics of the diamond drill rig used for drilling and sampling was adapted to operate the piezocone.

3.2.5 In situ Permeability Testing

Adjacent to Borehole NUTP-S3, an additional borehole was put down using NQ rods (internal diameter 60.3 mm) for the purpose of conducting falling head permeability tests. These tests were carried out at about 3 m intervals. Care was taken to ensure that the tailings was cleaned out from inside the drill rods. The tests were carried out by filling the drill rods with water and measuring the water level at frequent intervals. Conversion of the test data to average permeability of the tailings was carried out by the method described by Hvorslev (1948).

3.2.6 In situ Shear Strength Testing

Adjacent to Borehole NUTP-S3 shear strength tests were carried out at 1 m depth intervals using a self-boring Nilcon vane apparatus (Figure 3-3). In preparation for the execution of the Nilcon vane test, a hole was augered through the one metre thick ice cover on the tailings pond. The frame supporting the hydraulic head of the machine was then assembled, and put in place over the hole. The vane and slip coupling were then attached to a

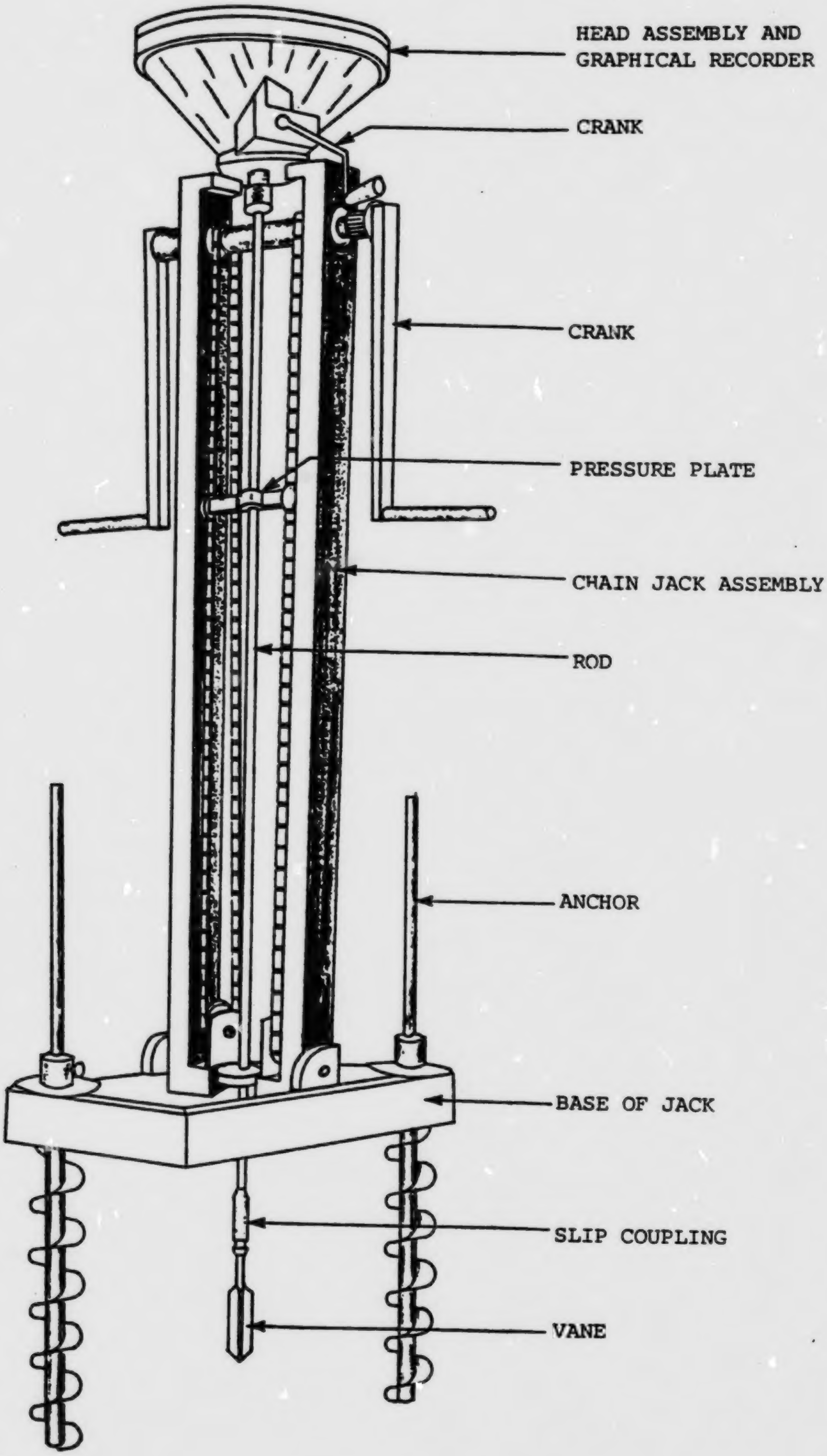

HEAD ASSEMBLY AND GRAPHICAL RECORDER
CRANK
CRANK
PRESSURE PLATE
CHAIN JACK ASSEMBLY
ROD
ANCHOR
BASE OF JACK
SLIP COUPLING
VANE

FIGURE. 3 - 3 NILCON VANE MOUNTED ON FRAME

length of rod and lowered in the hole. Additional rods were added until the surface of the tailings was encountered about 4 m below the ice surface.

The vane was then inserted to a depth of 1 m beneath the tailings surface where it was planned to conduct the first test. However, it was observed that at this depth the rods were still sinking quite readily under their own weight and that the tailings were not providing sufficient resistance to overcome the internal resistance in the slip coupling. This condition continued until a depth below the ice surface of 11 m was reached where the consistency of the tailings improved markedly, and the first test was started at that point.

The resistance of the tailings to the torque applied to the vane was recorded automatically by a mechanical plotter which traced the torque versus angular displacement of the vane. The springs of the plotter are calibrated so that the torque can be converted to shear stress on the soil around the vane.

Nilcon vane testing at the location of Borehole NUTP-S2 was abandoned after an initial attempt which encountered frozen tailings at the surface.

3.2.7 Field Laboratory

As part of this study a mobile soil mechanics laboratory was set up on site adjacent to the barium chloride treatment plant west of the tailings area. The field laboratory was equipped to carry out consolidation testing as well as routine index property tests including natural moisture content, unit weight, Atterberg limits, gradation analysis and specific gravity. Photographic records of 150 mm long sections of samples, split in half longitudinally, were also obtained.

The index property tests were carried out according to current ASTM procedures. An exception to this was the hydrometer test for determination of particle size distribution of the fine fraction of tailings samples. Standard procedures require that the sample be first washed with distilled water and then treated with hydrochloric acid to remove any carbonates. Carbonates in the soil would partly or totally offset the effect of dispersants, which are normally added in a concentration of 4 percent by weight of the soil-water suspension. In the present case, anticipated presence of water soluble salts such as gypsum as part of the tailings fabric, makes it undesirable to carry out this pre-treatment with distilled water. A preliminary series of tests was carried out to determine the optimum concentration of dispersant (sodium hexametaphosphate) which yielded the highest percentage of clay sized particles. As well, the effect of washing the sample with hydrochloric acid was studied.

A large soil sample obtained from the Borehole NUTP-S1 was thoroughly mixed to ensure homogeneity and then split into equal portions for testing to determine the optimum concentration of dispersant. All samples were washed with distilled water, with half the samples treated with acid. Varying amounts of dispersants were added to the samples and the tests run concurrently. The results indicated that washing with acid had an insignificant effect and that an eight percent concentration of dispersant yielded the highest observed percentage of clay sized particles for samples washed with distilled water only. All subsequent hydrometer tests were carried out using an 8 percent concentration of dispersant without prewashing the samples with hydrochloric acid.

Consolidation tests were carried out using the accelerated loading procedure suggested by Taylor (1948). With this procedure, a complete test could be completed in a single day. Minor modifications to a standard oedometer apparatus were incorporated to extend the test range to a very low initial stress of 4.5 kPa. This involves using a special set of weights and a holding collar on top of the standard oedometer ring. This collar consisted of a plastic ring with three set screws which held the loading cap just in touch with the top of the specimen and allowed setting up the loading arm and vertical displacement dial gauge without stressing the specimen. After the set up was completed, the test was initiated by retracting the set screws to release the loading cap and apply the first increment of loading to the specimen. Successive loadings were applied, using an incremental factor of 1.5, to a maximum of 250 kPa.

In addition to the routine soil mechanics laboratory tests, a specially fabricated pressure membrane apparatus was used to study the desaturation of the upper 10 m of tailings in Boreholes NUTP-S2 and S3. A schematic of the pressure chamber is shown on Figure 3-4. The procedure followed during the tests consisted of cutting a 75 mm long specimen of tailings and placing the specimen in the pressure chamber and increasing the air pressure in the chamber by a regulated compressed air source. Drainage of pore water was allowed from the bottom of the tailings specimen through the porous membrane which had an air entry value of 30 bars. Twice daily, the specimen was removed from the pressure chamber and weighed. This procedure was continued until the weight of the specimen became constant. The normal approach was then to repeat this procedure on the same specimen at progressively increased applied pressures up to the limit imposed by the air entry value of the porous membrane, to obtain the moisture content and void ratio versus suction (negative pore pressure) relationship for the sample. Due to time constraints however, it was necessary to carry out the test using different specimens obtained from the same sample, so that by using three pressure chambers, three different pressures could be applied concurrently. This procedure would lead to more scattering of the test results due to differences in the specimens. Field desaturated specimens were sealed with wax and preserved for strength determination in off site laboratory testing. Additional pressure membrane tests were also performed in offsite laboratories.

To obtain the void ratio of settled tailings for use in the computer model of self weight consolidation, nine sedimentation column tests were carried

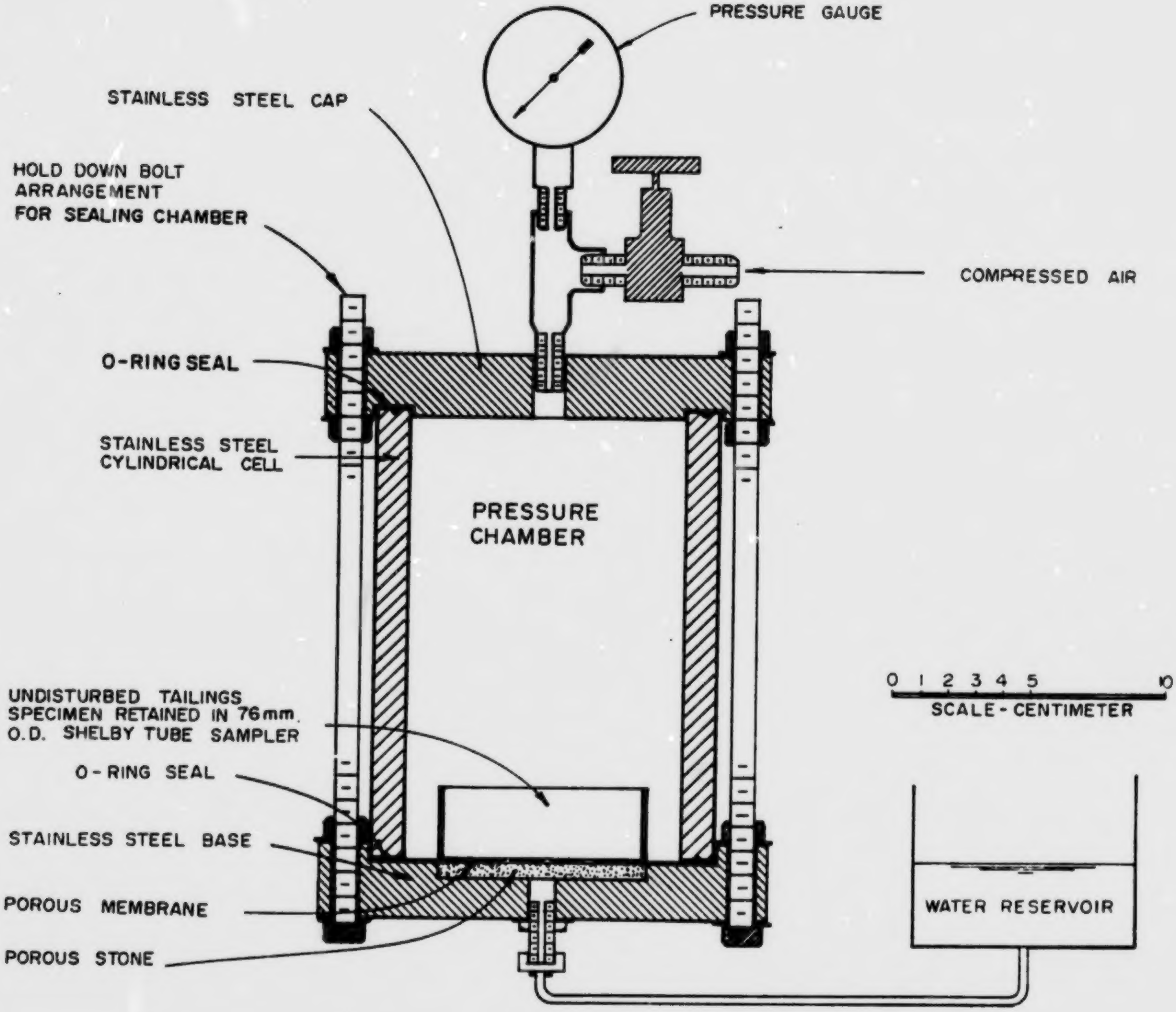

PRESSURE GAUGE
STAINLESS STEEL CAP
HOLD DOWN BOLT ARRANGEMENT FOR SEALING CHAMBER
COMPRESSED AIR
O-RING SEAL
STAINLESS STEEL CYLINDRICAL CELL
PRESSURE CHAMBER
0 1 2 3 4 5 10
SCALE - CENTIMETER
UNDISTURBED TAILINGS SPECIMEN RETAINED IN 76mm O.D. SHELBY TUBE SAMPLER
O - RING SEAL
STAINLESS STEEL BASE
POROUS MEMBRANE
POROUS STONE
WATER RESERVOIR

FIGURE 3 - 4 PRESSURE MEMBRANE TEST APPARATUS

out. These tests consisted of preparing a slurry of tailings solids and water at an initial solids content of 30 percent in a one litre graduated cylinder, and allowing sedimentation to take place. The void ratio of the settled tailings, e_{oo}, was computed from the bulk volume read from the graduated cylinder and the weight and specific gravity of the tailings solids.

3.2.8 Off-site Geotechnical Laboratory Testing

Additional pressure membrane tests were carried out in an offsite laboratory using similar procedures as in the field. An exception to this was that the specimen height was reduced to 25 mm so as to accelerate the test. Unconsolidated undrained (quick) triaxial tests were also carried out on desaturated specimens prepared in the field and offsite.

The presence of gypsum and/or bassinite (as detected in the mineralogical analysis discussed below) and other water soluble salts was believed to have adversely affected the results of the specific gravity and the grain size distribution tests carried out in the field. Consequently, additional check tests were carried out offsite using kerosene instead of water for the specific gravity tests.

Grain size distribution analyses were carried out by first washing the sample with warm distilled water to remove the soluble salts until no further weight loss occurred. By weighing the amount of solids before and after washing, the weight of soluble salts (predominantly gypsum) was recorded. This washing pretreatment generally took several days, at the end of which the hydrometer analyses were carried out using a concentration of dispersing agent, which appeared to be most effective. Two additional sedimentation tests were conducted offsite to supplement the tests carried out in the field. Identical procedures were employed for these tests.

3.2.9 Mineralogical Analyses

Two samples from Borehole NUTP-S2, Nos. 2 and 12, were delivered to the Saskatchewan Research Council (SRC) laboratories in Saskatoon in early March for mineralogical analyses. SRC carried out visual examination, X-ray diffraction and scanning electron microscopy for descriptive mineralogy and quantitative determination of gypsum and other forms of calcium sulphate and clay size minerals by particle size fractioning and chemical analyses.

A supplementary series of chemical/mineralogical analyses was subsequently commissioned by NUTP to examine:

- the leaching of major cations in slurry tests carried out for various time periods, and
- the composition and mineralogy of the samples before and after leaching.

The complete reports prepared by SRC are included in Appendix III of this report, and the reader is referred to them for a full discussion on the testing procedures employed and test results.

3.3 Results and Interpretation

3.3.1 Stratigraphy

Detailed logs of the boreholes are given in Appendix I of this report. Selected photographs of samples recovered from the boreholes are also shown following each borehole log in Appendix I. Abbreviated logs showing the principal features of stratigraphy encountered in the boreholes are shown in Figures 3-5A, 3-5B and 3-5C on the following pages.

Predominantly frozen tailings was encountered in Borehole NUTP-S1 (Fig. 3-5A) located in the north beach area, over the greater part of the depth of the borehole. Core recovery of up to 93 percent was obtained in the predominantly frozen zones. Occasional near horizontal ice lenses 10 to 50

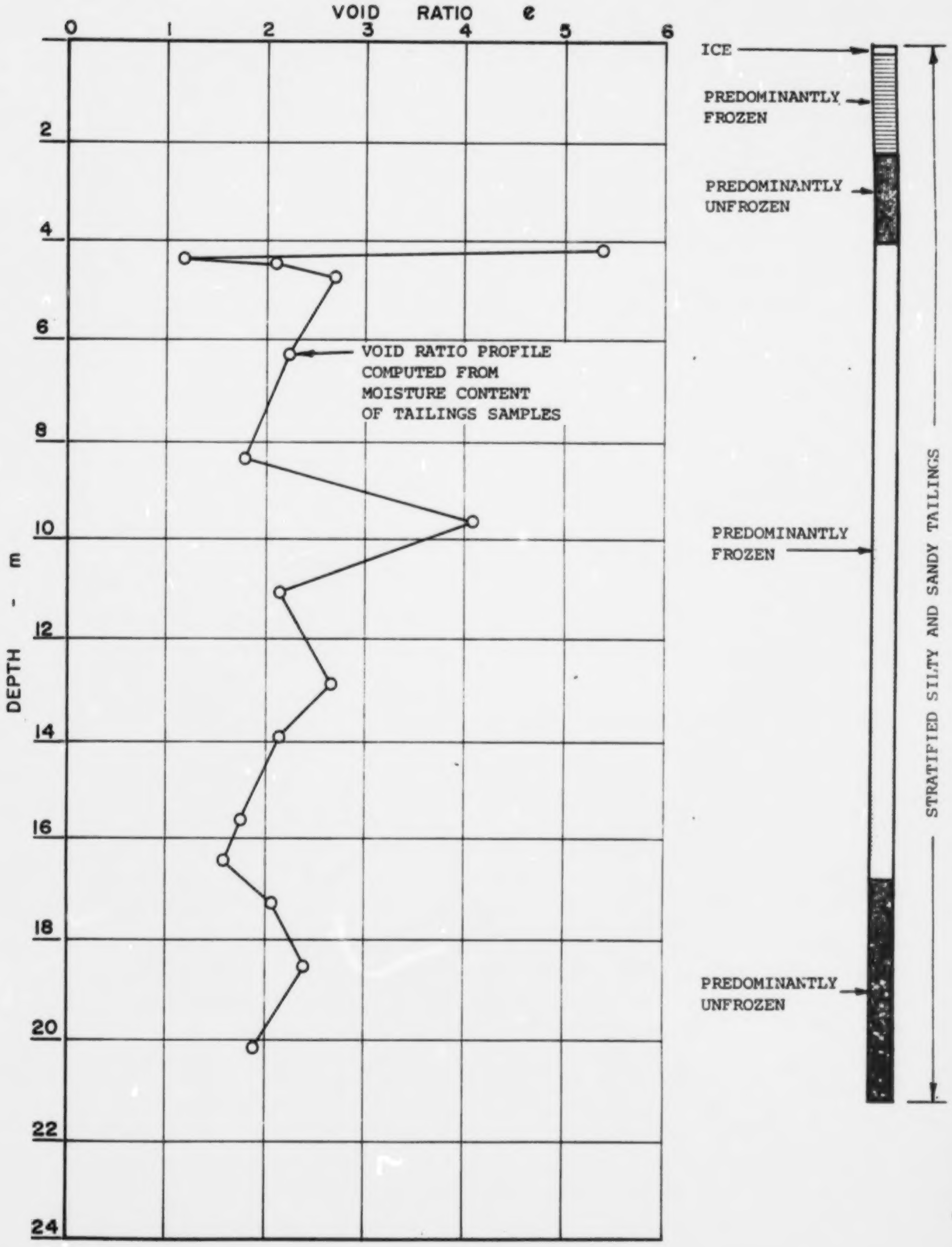


FIGURE 3 - 5A STRATIGRAPHY AND VOID RATIO PROFILE BOREHOLE NUTP - S1

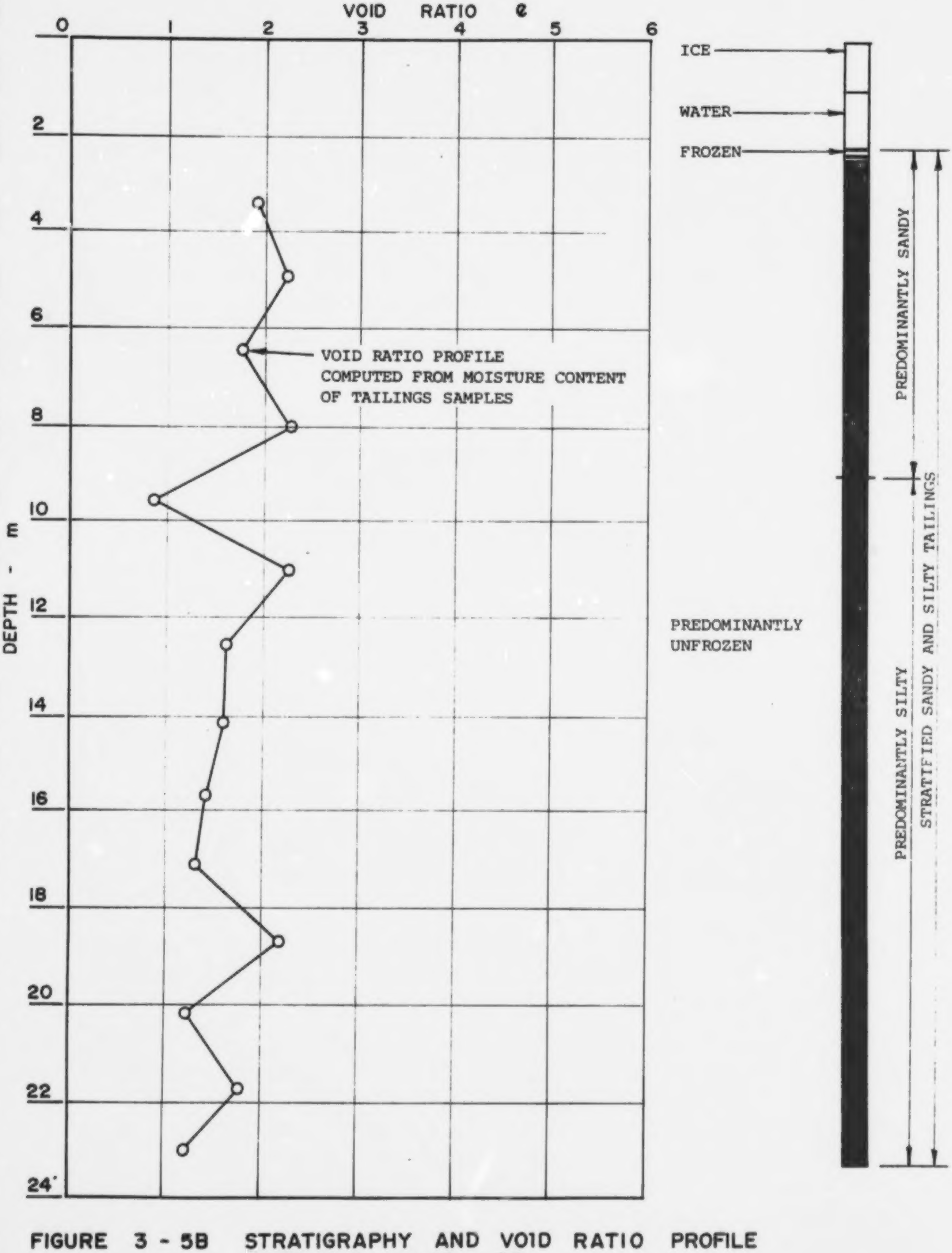


FIGURE 3 - 5B STRATIGRAPHY AND VOID RATIO PROFILE
BOREHOLE NUTP - S2

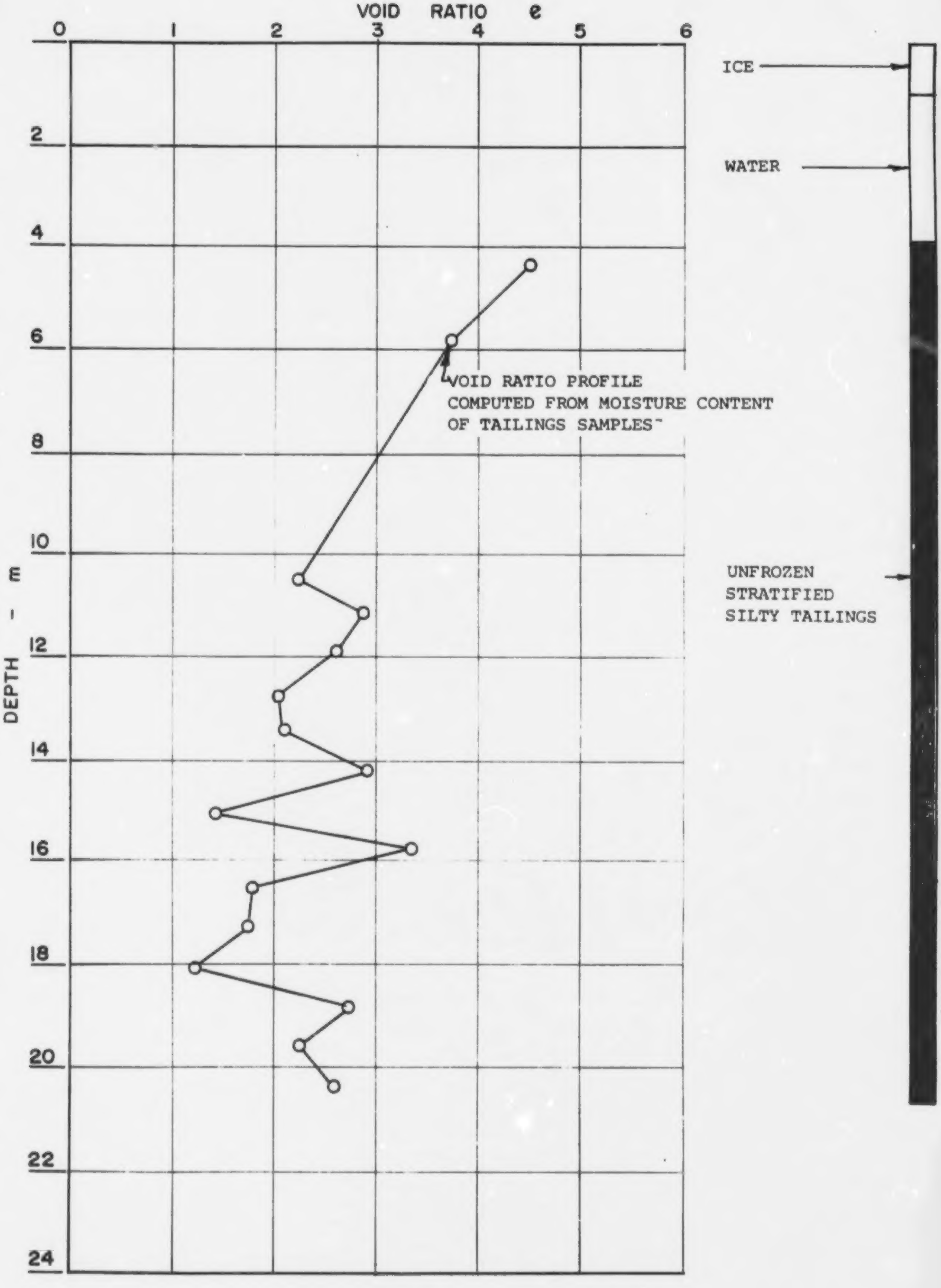


FIGURE 3-5C STRATIGRAPHY AND VOID RATIO PROFILE
BOREHOLE NUTP - S3

mm thick and near vertical ice filled fissures 5 to 10 mm thick were observed in the recovered cores. The tailings is visibly stratified, consisting of random layers of silty and sandy tailings. However, as discussed later in Section 3.3.2, the range of variation in grain size between different layers is not as great as the south beach area, reflecting the relatively simple history of deposition in this area.

At Borehole NUTP-S2 (Fig. 3-5B), located in the south beach area, the tailings surface occurred below 2.3 m of ice and water. The tailings was found to be essentially free from frozen zones except for an upper layer of 0.2 m below the surface. Stratification is pronounced with greater contrast in gradation between layers compared to Borehole NUTP-S1, as discussed in Section 3.3.2. There is also evidence that the tailings is predominantly sandy above a depth of 9 m (elevation 439.5 m) and predominantly silty below that level, which is consistent with the history of deposition in the area as described in Section 2.0 and illustrated in Figure 2-4.

At Borehole NUTP-S3 (Fig. 3-5C), located within the clarification pond, the tailings surface occurred below 4.0 m of ice and water. In-situ hydrometer tests showed an advanced stage of sedimentation with clear water of specific gravity at or close to 1.00 for nearly the full depth of the pond except a thin layer of slightly turbid water no more than 0.8 m thick above the tailings surface, where specific gravity values up to 1.10 were measured. The tailings at this location is predominantly silty (slimes) as anticipated from the history of deposition in this area. Pronounced stratification was also observed in samples recovered from this borehole.

3.3.2 Grain Size Distribution

As discussed in Sections 3.2.7 and 3.2.8, the grain size distribution of the fine fraction of tailings determined by hydrometer tests was found to be sensitive to procedures used in pretreatment by washing to remove soluble salts. A comparison of the contents of the clay sizes (minus 2 micron particles) determined from samples with and without washing by distilled water is shown in Table 3-2 below.

TABLE 3-2

EFFECTS OF PRETREATMENT BY WASHING WITH DISTILLED WATER

Borehole No.	Sample No.	% Minus #200 Mesh		% Minus 2 Micron	
		Not Washed	Washed	Not Washed	Washed
NUTP-S1	2*	71	31	6	8
NUTP-S2	1B	86	90	8	16
	2	92	87	18	21
	5*	14	79	0	20
	13	90	92	0	22
NUTP-S3	10	90	86	10	22
	11*	90	75	10	22
	15	90	97	8	19
	16	86	94	0	14

* Significant difference in minus #200 mesh fraction indicate basic difference in samples tested rather than effect of pretreatment.

Some of the samples (Borehole NUTP-S1 sample 2, Borehole NUTP-S2 sample 5 and Borehole NUTP-S3 sample 11) showed significant differences in the minus No. 200 mesh sizes between washed and unwashed tests as a result of actual variation in gradation within the sample due to the stratified nature of the tailings. Because the check tests using the washing pretreatment were carried out in offsite laboratories on portions of the samples which were different from those carried out initially in the field, such changes in gradation between the test specimens could not be avoided. However, from comparative tests on those samples which showed relatively consistent contents of minus No. 200 mesh fractions between the specimens with and without pretreatment, it can be seen that the pretreatment did consistently increase the measured content of clay sizes (minus 2 micron sizes) although the extent of the apparent increases are quite variable.

Although the tests carried out in the field without pretreatment by washing are subject to possible errors in the clay contents, the measured contents of minus No. 200 sizes are quite reliable and provide a useful indication of the occurrence of sandy or silty tailings at various locations in plan and in depth. For this purpose, detailed results of all grain size analyses, with or without pretreatment by washing, are shown in Tables I-1A, I-1B and I-1C in Appendix I of this report. Overall ranges and the frequency of occurence of silty and sandy samples in each borehole are shown in Figures 3-6A, 3-6B and 3-6C on the following pages.

Besides verifying the stratified nature of the tailings, the gradation analyses also confirmed the expected trend of areal segregation from the history of deposition in the area. Thus, Borehole NUTP-S1 showed (Figure 3-6A) an overall range close to the average grind of the tailings before deposition. Relatively minor segregation into sandy and silty layers is consistent with the proximity of the area to the principal discharge point. Freezing in winter in this area also tended to reduce the degree of segregation. Borehole NUTP-S2, located in an area with a complex history of deposition as described earlier, with discharge of cyclone overflows in the early stages and intermittent point discharge from the south dam in the later stages, exhibits the large range of gradation as shown in Figure 3-6B. Examination of the individual test results listed in Table I-1B, Appendix I, indicate the more sandy nature of the upper part of the tailings at this locations, above a depth of about 9 m or elevation 439.5 m, as anticipated from the zoning shown in Figure 2-4. At Borehole NUTP-S3, all samples tested consist of silty tailings (slimes) with a minimum measured content of about 67 percent of minus No. 200 mesh sizes and up to a measured maximum clay content (minus 2 micron sizes) of about 33 percent. This is consistent with the location of the borehole in the central part of the clarification pond area.

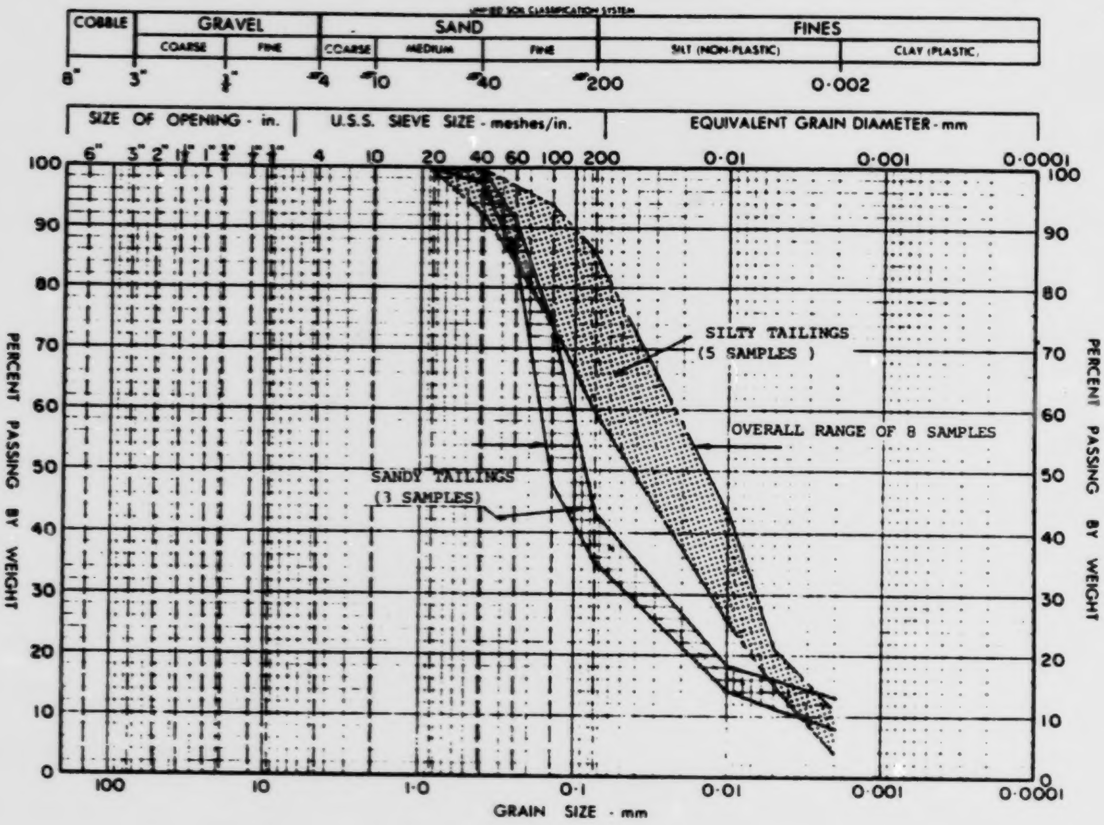


FIGURE 3-6A GRAIN SIZE DISTRIBUTION
BOREHOLE NUTP- SI

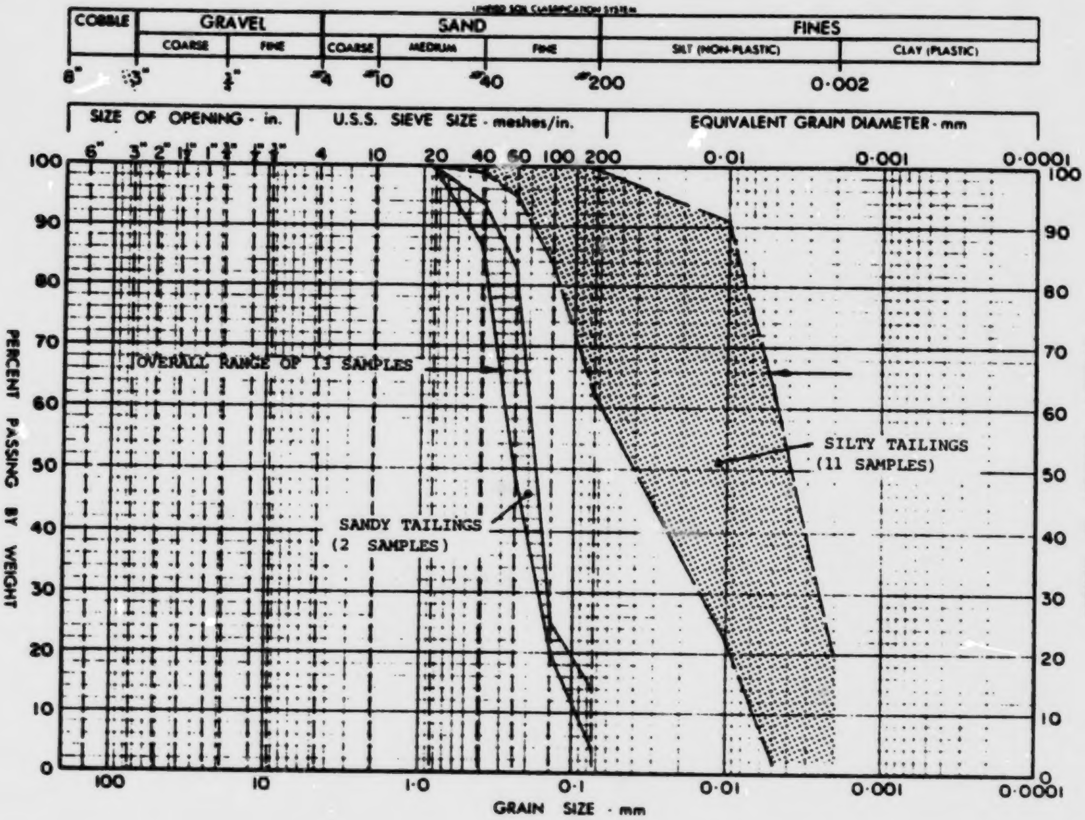


FIGURE 3-6B GRAIN SIZE DISTRIBUTION
BOREHOLE NUTP - S2

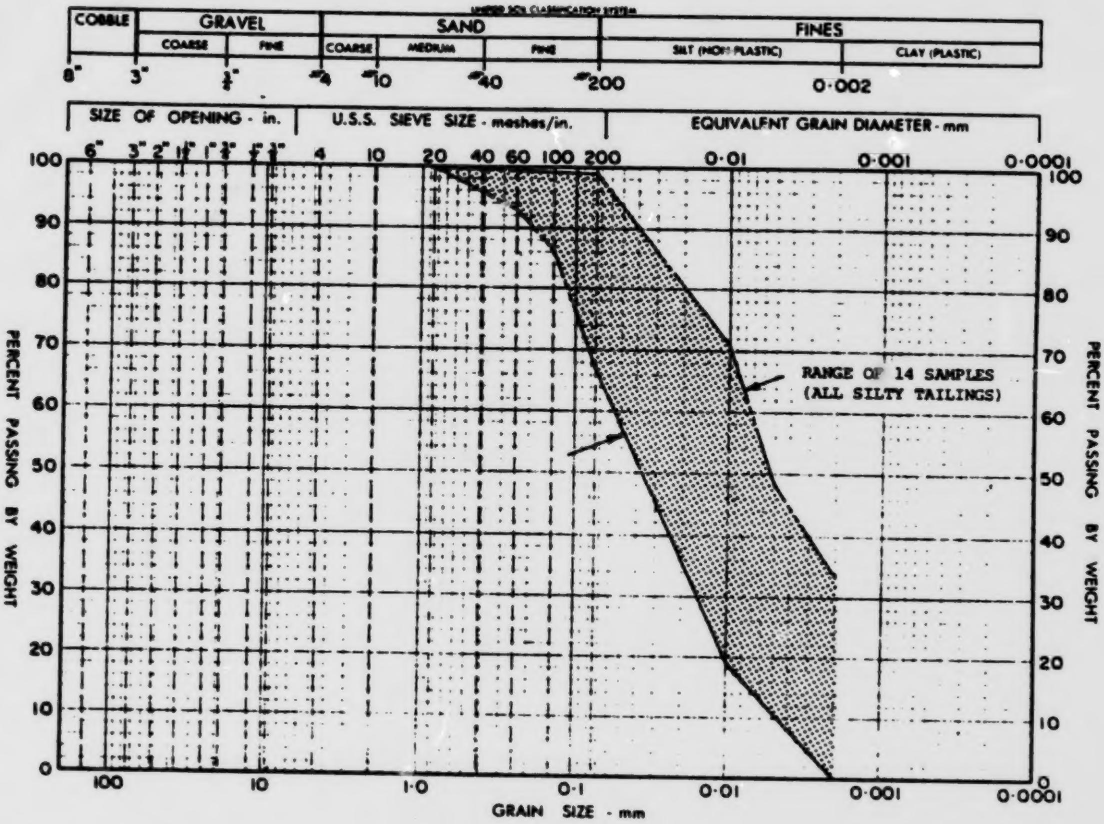


FIGURE 3-6C GRAIN SIZE DISTRIBUTION
BOREHOLE NUTP - S3

The care taken in the present investigation to maximize dispersal of the clay sized particles during hydrometer analyses, revealed much higher clay contents than previous reported (Gulf Minerals, 1981b). The silty tailings samples have clay contents generally in the range of 10 to 20 percent, with a maximum of 33 percent as indicated above. This is in general agreement with the mineralogical test work carried out by Dr. R.G. Arnold of the Saskatchewan Research Council, with measured minus 5 micron size fractions of 22 percent on two test samples. These clay contents are adequate to account for the high void ratios and compressibility of the tailings as observed in place and in oedometer testing.

3.3.3 Atterberg Limits

Atterberg limits obtained on silty tailings samples are shown in Tables I-1A, I-1B and I-1C in Appendix I and plotted in the plasticity charts shown in Figure 3-7 on the following page. The ranges and averages are tabulated below:

	Liquid Limit		Plastic Limit	
	Range	Average	Range	Average
Borehole NUTP-S1	50-61	58	41-46	43
Borehole NUTP-S2	52-85	70	38-67	53
Borehole NUTP-S3	58-106	75	37-71	52

These results again reflect the pattern of areal segreation, and classify the silty tailings as compressible inorganic silts of high plasticity.

3.3.4 Moisture Content, Bulk Density and Specific Gravity

Moisture contents of the tailings in place are shown plotted against depth in the borehole logs in Appendix I. These values are also shown together with bulk density and specific gravity values in Table I-1 in Appendix I. The values of specific gravity are those determined by using kerosene in

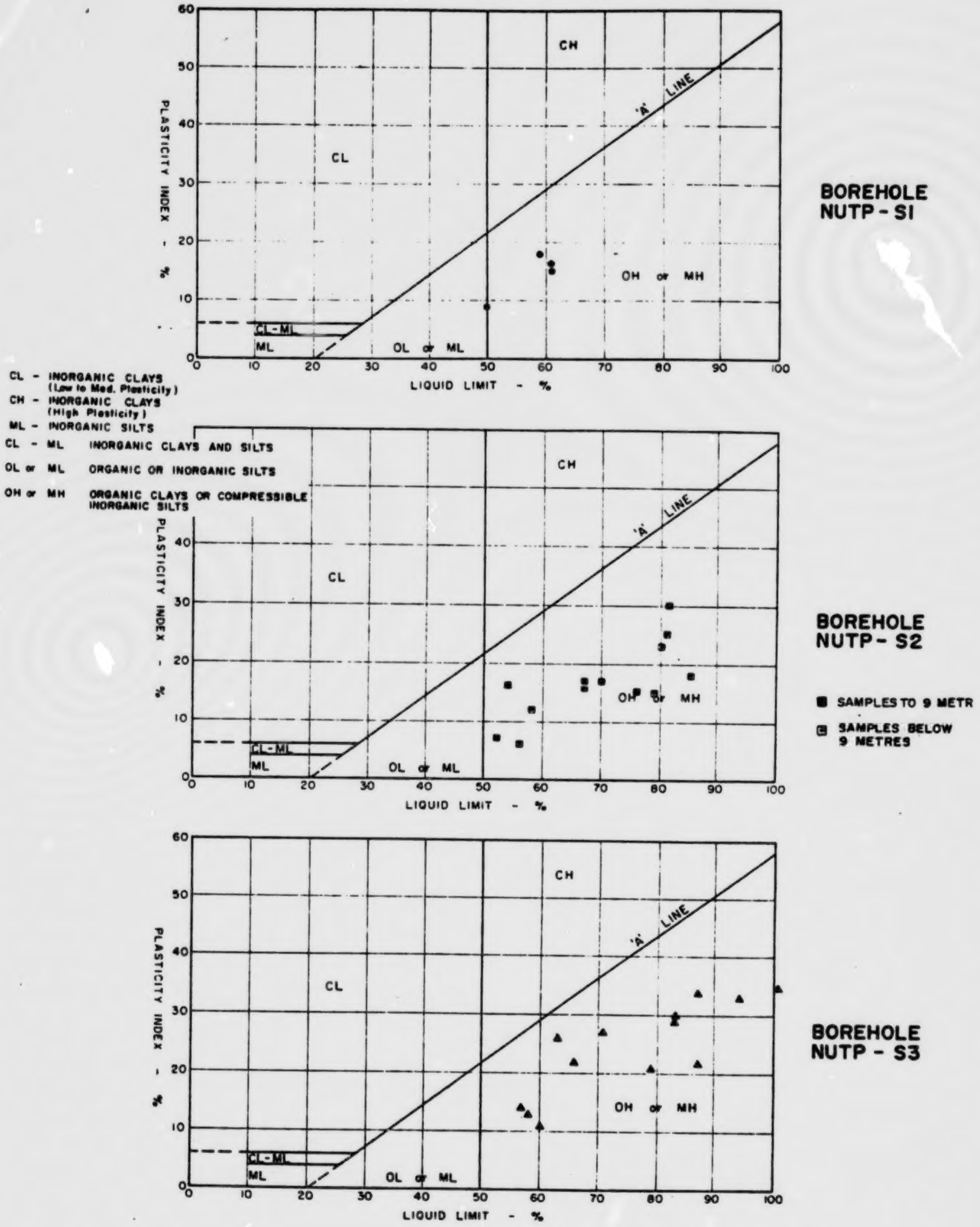


FIGURE 3-7 PLASTICITY OF SILTY TAILINGS

off-site laboratory testing. An initial series of tests carried out in the field laboratory using distilled water gave erroneous values due to dissolution of gypsum and other water soluble salts within the tailings, and those values are not reported.

	Moisture Content %		Bulk Density (kN/m^3)		Specific Gravity of Solids	
	Range	Average	Range	Average	Range	Average
Borehole NUTP-S1	50.9-207.6	93.3	-	-	-	-
Borehole NUTP-S2	33.8- 88.5	66.0	14.4-17.2	15.7	2.54-2.61	2.58
Borehole NUTP-S3	47.5-175.3	98.1	12.2-17.3	15.3	2.53-2.69	2.61

The high average moisture content at NUTP-S1 compared to that at NUTP-S2 reflect the entrained ice and arrested process of consolidation due to freezing of the tailings. The high average moisture content at NUTP-S3 reflects the predominance of silty tailings (slimes) and consolidation under submerged condition in the clarification pond area.

3.3.5 Void Ratio and Compressibility

In-situ void ratio profiles at the boreholes are shown in the borehole logs in Appendix I and in Figures 3-1A, 3-1B and 3-1C in Section 3.3.1. These void ratio (e) values were computed from moisture contents (w) of samples and average specific gravity of solids(g_s), assuming the tailings to be fully saturated, as given in the following relationship.

$$e = w\ g_s$$

Detailed results of oedometer tests on selected samples are given in Figures I-1 to I-12 in Appendix I. The compressibility curves generally show the effects of self-weight consolidation with preconsolidation pressures in the range of 10 to 100 kPa. General trends of decreasing permeability with decreasing void ratio are exhibited by all test samples. Ranges and means of compression index values are as follows:

	Compression Index, C_c	
	Range	Average
Borehole NUTP-S1	-	-
Borehole NUTP-S2	0.16-0.95	0.53
Borehole NUTP-S3	0.27-1.45	0.81

The large spread in compression index values relect the stratified nature of the tailings and the higher average value at NUTP-S3 compared to NUTP-S2 indicate a higher concentration of fines in the area of the clarification pond as anticipated.

Initial void ratios under practically zero stress condition in sedimentation column tests are shown in Tables I-1B and I-1C in Appendix I. These range from 2.26 to 5.04 with an average of 3.84, and again reflect the variability of the tailings.

3.3.6 Desaturation Tests

Desaturation tests were carried out using the specially fabricated pressure membrane apparatus, to obtain information on the limit void ratios, "e_{SL}" and "e_{DL}" on desiccation. The moisture content versus degree of saturation characteristics are shown in Figures I-13 and I-14 in Appendix I. These indicate desaturation to start from moisture contents of 110% and 98.4%, respectively. As desaturation progresed under increased suction (by increasing applied cell pressures in the pressure membrane apparatus) shrinkage of the test specimens were accompanied by cracking.

Gross void ratios (including volume of shrinkage cracks) determined by measuring the reduction in height of the samples, are shown plotted against degree of saturation in Figures I-15 and I-16, Appendix I. The gross void ratio at zero percent of saturation in each case was determined from the oven-dried specimen. From these test characteristics, the values of the saturation limit e_{SL} were estimated to be 2.85 and 2.50, respectively. The gross void ratio at zero percent of saturation gives an approximate indication of the lower bound value of the desiccation limit "e_{SL}", and the measured values are about 2.41 and 2.14, respectively.

3.3.7 Piezocone Testing

The complete series of computer plots of various parameters measured during testing by using a piezocone at CPT1 and 1A adjacent to Borehole NUTP-S2 and CPT2 adjacent to Borehole NUTP-S2, are shown in Appendix II together with interpretation of the data, as prepared by Dr. R.G. Campanella of the University of British Columbia. The measured basic parameters include dynamic pore pressures (in excess of those at the tailings surface), cone bearing and sleeve friction, with derived parameters designated differential pore pressure ratio and friction ratio used in interpretation of the test data. Significant information obtained from the interpretation of these measured parameters include soil classification and undrained shear strength tabulated at 0.5 m intervals in Appendix II. It is of interest to note that the upper 9.5 m in CPT-1A was described as "sensitive fine grained" as distinct from the lower part which was described predominantly as "clayey silt to silty clay", which in general terms, supports the interpretation based on grain size analyses of samples from the adjacent Borehole NUTP-S2, as discussed in Section 3.3.2 . The interpretation from CPT2 however, does not correlate quite as well with the results of grain size analyses on samples from the adjacent Borehole NUTP-S3. Dr. Campanella noted however, that the piezocone data is "at the limit of its resolution and, actually, beyond the limit of its accuracy".

Undrained shear strengths interpretated from the piezocone test data exhibit an increasing trend with depth, (Figures 3-8 and 3-9) ranging from about 15 kPa to 64 kpa in CPT 1-1A and from near zero to 31 kPa in CPT 2. A high value of 105 kPa at the end of CPT 2 is believed to be in or influenced by natural ground at that level and therefore is not representative of the strength of tailings at that point. Somewhat higher strengths measured in CPT 1 near the surface of the tailings reflect the more sandy nature of the tailings there as anticipated from the history of deposition in this area.

Some difficulties were encountered in interpreting the dissipation tests in determining in-situ values of coefficient of consolidation "c_v" and thereby coefficient of average permeability. The problem apparently arose from drift of the electronic sensing equipment probably due to temperature changes during the test period, causing some pore water pressure readings to fall below the hydrostatic values at a number of test depths. If no drift occurred during any particular test, the interpretation of the rate dissipation of excess pore water pressure would not be adversely affected by a constant error due to drift occurring prior to that test. Based on this assumption, the computed permeability is shown plotted in Figure 3-10. These values range from 2.6×10^{-5} to 2.2×10^{-4} cm/sec. in CPT 1-1A and from 1.5×10^{-4} to 8.5×10^{-4} cm/sec. in CPT 2. General trends of decreasing permeability with depth are also evident as anticipated. It should be noted that each of these interpreted permeability values represent, theoretically speaking, the geometric mean of horizontal and vertical permeability values of an unknown volume of tailings around the test location. Stratification and spatial variation of gradation of the tailings further complicate the interpretation of these tests.

3.3.8 In-situ Vane Shear Tests

The undrained shear strengths of the tailings adjacent to Borehole NUTP-S3 as determined by using the Nilcon Vane are also shown plotted in Figure 3-9

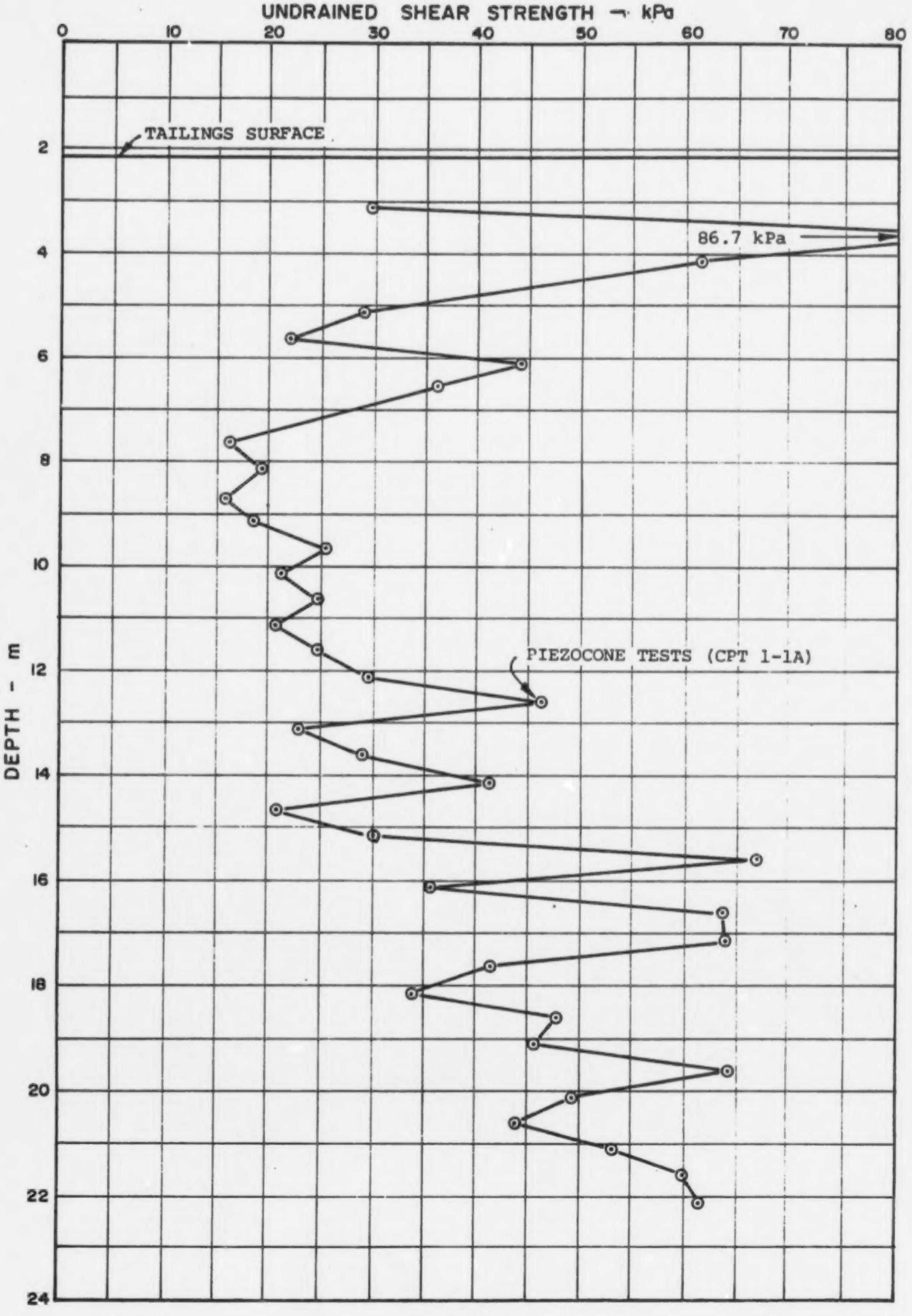


FIGURE 3- 8 UNDRAINED SHEAR STRENGTH - BOREHOLE NUTP - S2

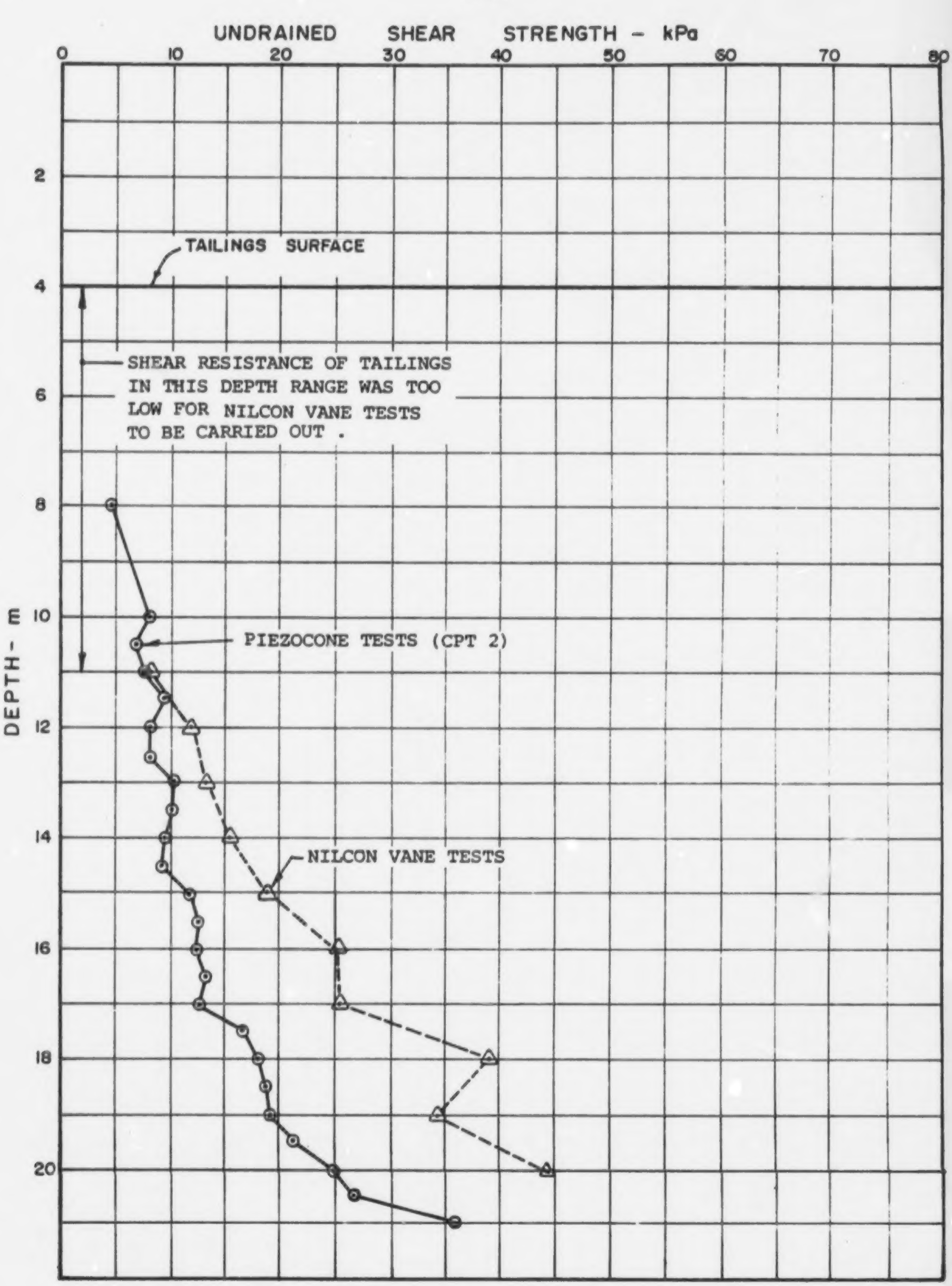


FIGURE 3-9 UNDRAINED SHEAR STRENGTH - BOREHOLE NUTP-S

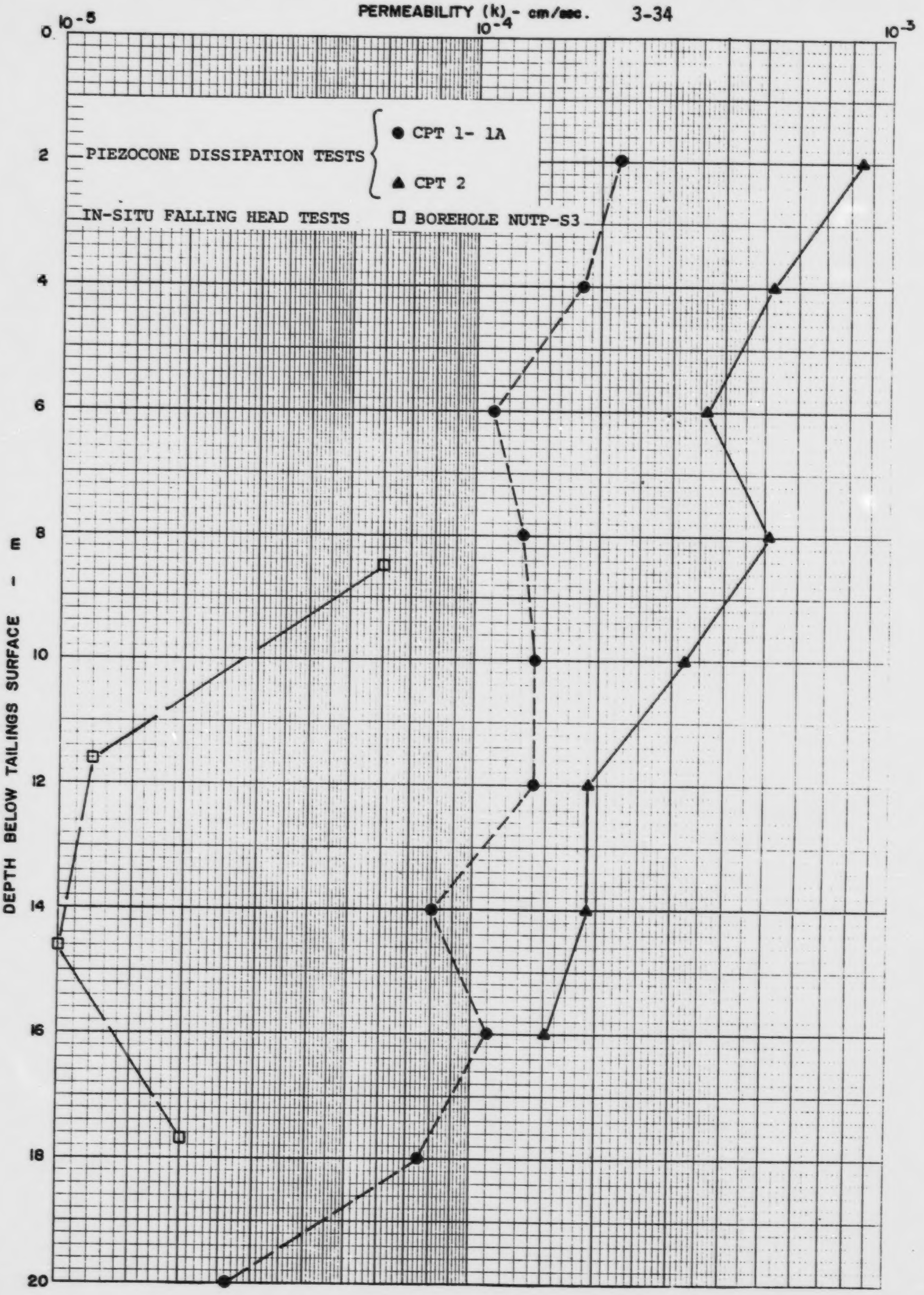


FIGURE 3 - 10 IN SITU PERMEABILITY OF TAILINGS

for comparison with the piezocone test results. These show reasonably good agreement in general trend, with slightly higher strengths measured by the vane.

3.3.9 In-situ Faling Head Tests

Falling head tests carried out in a borehole adjacent to NUTP-S3 gave field permeability values ranging between 1.0 x 10^{-5} and 6.9 10^{-5} cm/sec. These represent the geometric mean values of vertical and horizontal permeabilities as in the case of the piezocone test values, and are plotted also in Figure 3-10 for comparison purposes.

3.3.10 Piezometric Observations

Due to problems with freezing of the water within the standpipes of the piezometers, even with the use of a special antifreeze solution for filling the standpipes during installation, accurate stabilized piezometric water levels could not be obtained during the field work. Readings taken from those piezometers installed in and adjacent to Boreholes NUTP-S2 and NUTP-S3 before freezing occurred, generally indicate water levels within 50 mm of the pond water level except two piezometers in Borehole NUTP-S3 located at tip depths of 19.8 m and 12.2 m, which recorded water levels at 0.8 m and 0.4 m below pond water level, respectively.

At borehole NUTP-S1 the tailings took a considerable length of time to close in on the piezometer tips which were installed without special backfilling of the borehole, due to the frozen nature of the tailings at this location. Thus in filling the piezometer located at 21.2 m depth with antifreeze solution shortly after installation, free flow of the solution into the open hole through the piezometer tip occurred, preventing build up of the water level within the standpipe. To avoid using an excessive quantity of antifreeze this filling operation was not continued. Consequently, water levels measured within this piezometer remained at about

11.3 m below pond water level over a period of 2 to 3 weeks after installation. This is believed to be a false reading due to a limited amount of antifreeze introduced into the bottom of the open borehole which might have become frozen in subsequently and hydraulically cut-off from any major body of unfrozen tailings with real pore water pressures. This reading is not considered to be the result of undrainage effect due to the proximity of the piezometer to the North Dam unless proven to be so with additional piezometer installations in the vicinity, or from response tests carried out in it under summer conditions. The other two piezometers installed in this borehole were both frozen shortly after installation, with water levels at pond water level.

4.0 POSSIBLE DECOMMISSIONING STRATEGY

It is not the intent of this study to design the decommissioning measures in detail for the Rabbit Lake tailings deposit. However, as discussed in Part A of the study, decommissioning schedule and strategy would have an important influence on the long term cover settlements. In this context only therefore, these aspects are examined here to provide guidance to the mathematical modelling of the consolidation/desiccation behaviour of the tailings deposit described in Section 5.0.

Various methods for promoting consolidation of an existing tailings deposit such as the Rabbit Lake deposit, to minimize long term settlements after decommissioning, were appraised in Part A of the study. It was concluded that a conceptual scheme of deferred cover construction accompanied by suitable surface drainage measures to take advantage of the effects of desiccation by natural evaporation, appeared to be a comparatively cost effective and technically simple method for reclaiming such a facility. With the site specific information obtained from the investigation, more definitive analyses of the long term settlements for the Rabbit Lake tailings deposit are carried out as described in Section 5.0, to further investigate the viability and merits of such a scheme.

Because the effectiveness of the scheme depends on the extent to which pre-induced settlements by desiccation are allowed to take place prior to cover construction, time scheduling of decommissioning activities is one of the most important considerations in planning the work. Disposal in the tailings impoundment area under this study was completed in August, 1985. It is possible, however, that this facility will continue to be used for temporary storage of effluent water from the B-zone milling operation for some time before decommissioning of the facility will begin. For present purposes, it is assumed that decommissioning will begin two years after the end of tailings deposition, or in August, 1987. At that time, the clarification pond can be drained and desiccation of the tailings within that area, which is most susceptible to large settlements, can then be allowed to take place to pre-induce settlements.

A conceptual arrangement for drainage of the clarification pond, using the existing pump barge, was described in the Part A report. Placement of the sand blanket over the tailings surface, shown in Figure 8-1 of that report, could be carried out prior to lowering of the clarification pond by sluicing in sand as hydraulic fill and allowing it to settle through water. Controlled placement of the sand fill in this manner in thin uniform lifts of say about 0.2 m each and allowing a suitable length of time between lifts for the underlying tailings to consolidate, will prevent punching of the sand fill through the tailings surface. Placement of a filter fabric over the tailings surface initially before placing the sand could also be considered as an additional measure against local punching of the sand fill into the tailings. Alternatively, the sand blanket construction could be carried out in winter. In this case lowering of the clarification pond from the pump barge would be carried out to the extent possible in the fall. The pump barge would then be removed prior to freeze up. Placement of the sand fill as well as an access road of granular fill from the shoreline, could be carried out on the frozen surface of the tailings provided care is taken to check that sufficient depth of frost penetration is available before starting the fill placement. Suitable measures such as controlled placement to ensure uniform fill thickness and gentle side slopes around the perimeter of the sand blanket would have to be taken to prevent foundation failure after spring thaw. Placement of a filter fabric at the base of the fill in this case could also be considered to provide further safeguard against local punching failure of the fill. In both cases, it is believed that considerable care could be exercised in placement of the sand as required without major expense due to the very small quantities of fill involved.

As discussed in Part A, the success of the proposed decommissioning strategy will rely on the effectiveness of surface drainage during the period of desiccation of the tailings prior to cover placement and minimization of further evaporative losses from the tailings once the cover is in place. To maximize the rate and depth of desiccation before cover construction, trenching of the tailings surface to promote drainage of water in shrinkage cracks formed in the tailings and periodic harrowing to turn over the surficial layer of tailings and expose the underlying wet tailings to further evaporation, would be desirable.

Haliburton (1979) indicated the successful use of an amphibious vehicle called the Riverine Utility Craft (RUC) in trenching of a dredged spoil surface during the initial stage of desiccation before a sufficiently thick and competent crust was developed to support conventional equipment (Figure 4-6). The use of a dragline on suitable pads was found to be feasible after a 0.3 m crust was established. In the present instance, advantage could again be taken of the cold climate to enable initial trenching to be carried out, using conventional equipment such as a track mounted backhoe on the frozen tailings surface in winter. Careful timing of the trenching operation either in early winter or early spring when the frozen layer is sufficiently strong to support light equipment but not too thick and hard to break through with conventional excavating tools, could optimize this operation. A special carbide tipped cable trencher, used with a certain measure of success in trenching through frozen till fill for extending a soil-bentonite liner of a tailings impoundment dam at the Amok Ltd. Cluff Lake project, could also be considered for this purpose. After an initial desiccated crust is established, a disc harrow or other similar equipment towed by a wide track bulldozer or other low ground pressure equipment, could be used to turn over the tailings to promote evaporation.

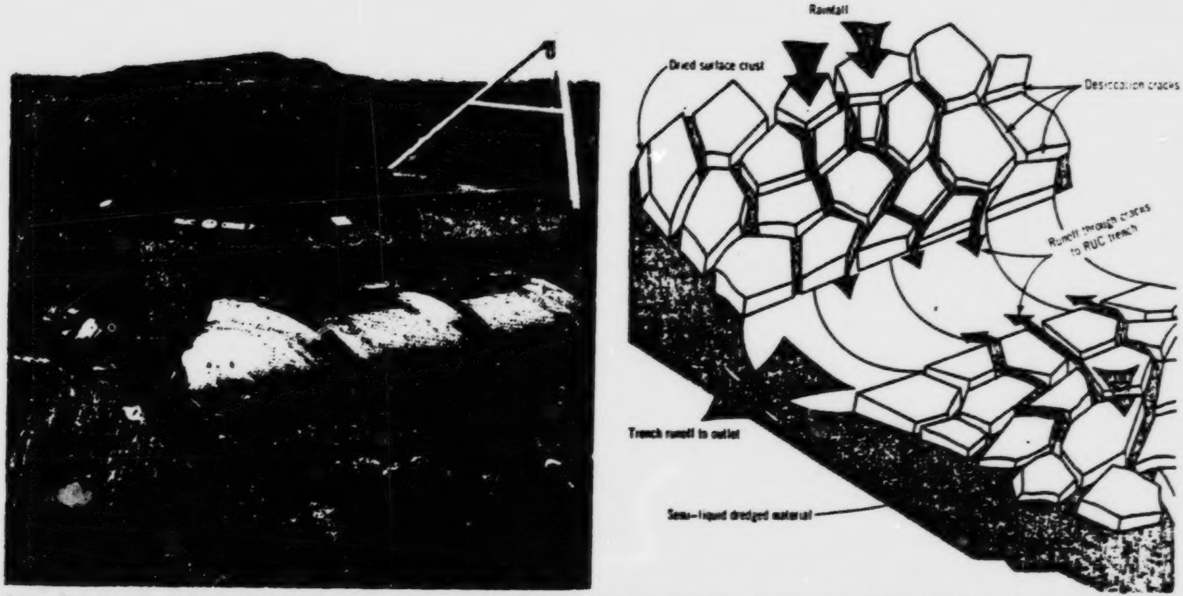


Fig. 1. Progressive trenching method of dewatering dredged material. Removing ponded surface water and precipitation from disposal area allows nature to desiccate dredged material inexpensively. Riverine Utility Craft (left), amphibious twin Archimedean screw propulsion vehicle originally developed by U.S. Marine Corps for reconnaissance in South Vietnam, was only vehicle identified which could conduct initial trenching in disposal areas with extremely low soil support capacity.

As RUC tranverses disposal area, rotors break through thin desiccation crust and create pair of semi-circular ruts which may be used as drainage trenches. Ponded surface water and precipitation drain down RUC trenches to outflow weirs and off site (right). Once surface water is removed, evaporative drying can occur.

Once initial RUC trenches are made throughout disposal area and ponded surface water removed, dredged material will desiccate into crust with consistency of normal soil. When desiccation cracks occur in bottom of RUC trenches, another RUC pass down existing RUC trenches should be made to deepen them and allow continued drying of underlying dredged material. Once several RUC trenching cycles have been carried out, enough crust thickness is developed to support conventional draglines on mats, which then deepen RUC trenches and allow evaporative drying to continue.

FIGURE 4-6 USE OF ALL-TERRAIN VEHICLE FOR TRENCHING TO PROMOTE SURFACE RUNOFF
(HALIBURTON, 1979)

5.0 MATHEMATICAL MODELLING

5.1 Overview

The history of deposition described in Section 2-2 indicates that the Rabbit Lake tailings deposit can be divided longitudinally (in the north-south direction) into three main zones with two transition zones between them as shown in Figure 2-4.

The clarification pond area and the south beach area with the transition between them were found to be essentially free from frozen tailings below the active layer (within the frost depth of penetration in the area of some 3 m from the surface) which is subjected to seasonal freeze-thaw cycles. Mathematical modelling of the consolidation/desiccation behaviour of unfrozen tailings, using the computer program PCDDF as discussed in detail in Part A, is applicable to these areas. The two boreholes (NUTP-S2 and -S3) put down in this investigation enabled one-dimensional models to be developed to characterize the two different areas (south beach and clarification pond). In addition, information from Borehole 81-T3 put down in a previous investigation (Gulf Minerals 1981b) provided information to characterize the transition area between the clarification pond and the south beach. Variations in detail within each of these areas are of course to be anticipated. Definition of such variations is beyond the scope of this study. Two dimensional settlement profiles presented in Section 5.6, developed from the one-dimensional models, are therefore essentially of a illustrative, rather than definitive, nature and should be taken in that context only.

The north beach area was found to be underlain by predominantly frozen tailings throughout the full depth of the deposit. Little or no surface settlements are anticipated to occur in this area as long as the tailings remain frozen as permafrost and thus no settlement analyses are required under this condition. However, the eventuality of the tailings thawing out completely in the future, although highly unlikely, is considered. There are no known methods for predicting the magnitude and rate of settlement resulting from such a phenomenon. However, a crude estimate of the order of magnitude of possible ultimate settlements if the tailings did thaw out completely, is presented to give some indication of the possible consequences of such an eventuality.

The computer program PCDDF simulates the consolidation/desiccation process from the beginning of deposition of the consolidating material (tailings) and therefore requires input of the initial properties of the unconsolidated material at the time of deposition. As a result of this constraint the in-situ properties of the tailings at the time of the investigation described in Section 3.0, cannot be used directly for input as initial state parameters in predicting future consolidation/desiccation effects, even though they represent the true initial conditions of the tailings relative to these long term processes. Instead, the information will be used for comparison with the results of analyses simulating the self-weight consolidation process during tailings deposition which had taken place prior to the investigation referred to above and thus it serves to validate or to calibrate the mathematical model. An iterative process can be followed to improve the simulation by adjusting the input parameters, if necessary. In this context therefore, modelling of the self-weight consolidation of the Rabbit Lake tailings during deposition is required and the results are presented in Section 5.2.

Modelling of desiccation/consolidation effects during the progress of decommissioning is described in Section 5.3 and that of post-decommissioning settlement, in Section 5.4

5.2 Consolidation During Deposition

Three one-dimensional models are studied in this section. These consist of a vertical column of tailings in each of the following areas:

- the clarification pond area, represented by Borehole NUTP-S3
- the south beach area, represented by Borehole NUTP-S2
- the transition between the above areas, represented by Borehole 81-T3

5.2.1 Input Data

The information required for input to the analyses by the program PCDDF include compressibility in the form of e-σ' relationships, permeability in the form of e-k relationships, the initial void ratio "e_{oo}" under zero stress and the rate of accretion of the tailings expressed in terms of unconsolidated "Lagrangian" height of the tailings as a function of time.

Compressibility

The compressibility information from in-situ void ratio profiles at the above indicated boreholes and from oedometer tests are shown in the following figures:

Clarification pond area (Borehole NUTP-S3) - Figure 5-1A

South beach area (Borehole NUTP-S2) - Figure 5-1B

Transition area (Borehole 81-T3) - Figure 5-1C

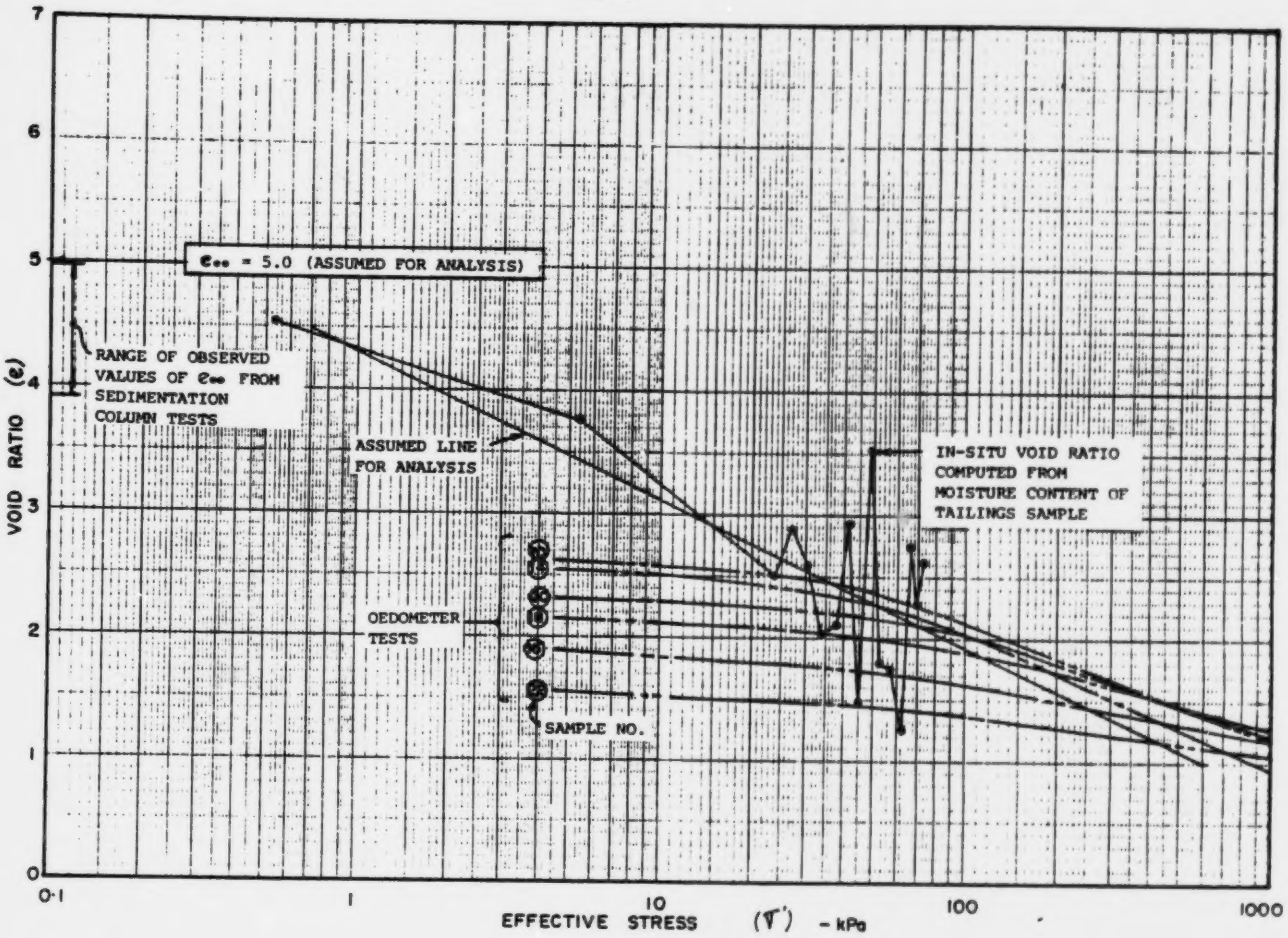


FIGURE 5 - 1A e - σ' RELATIONSHIP
BOREHOLE NUTP - S3

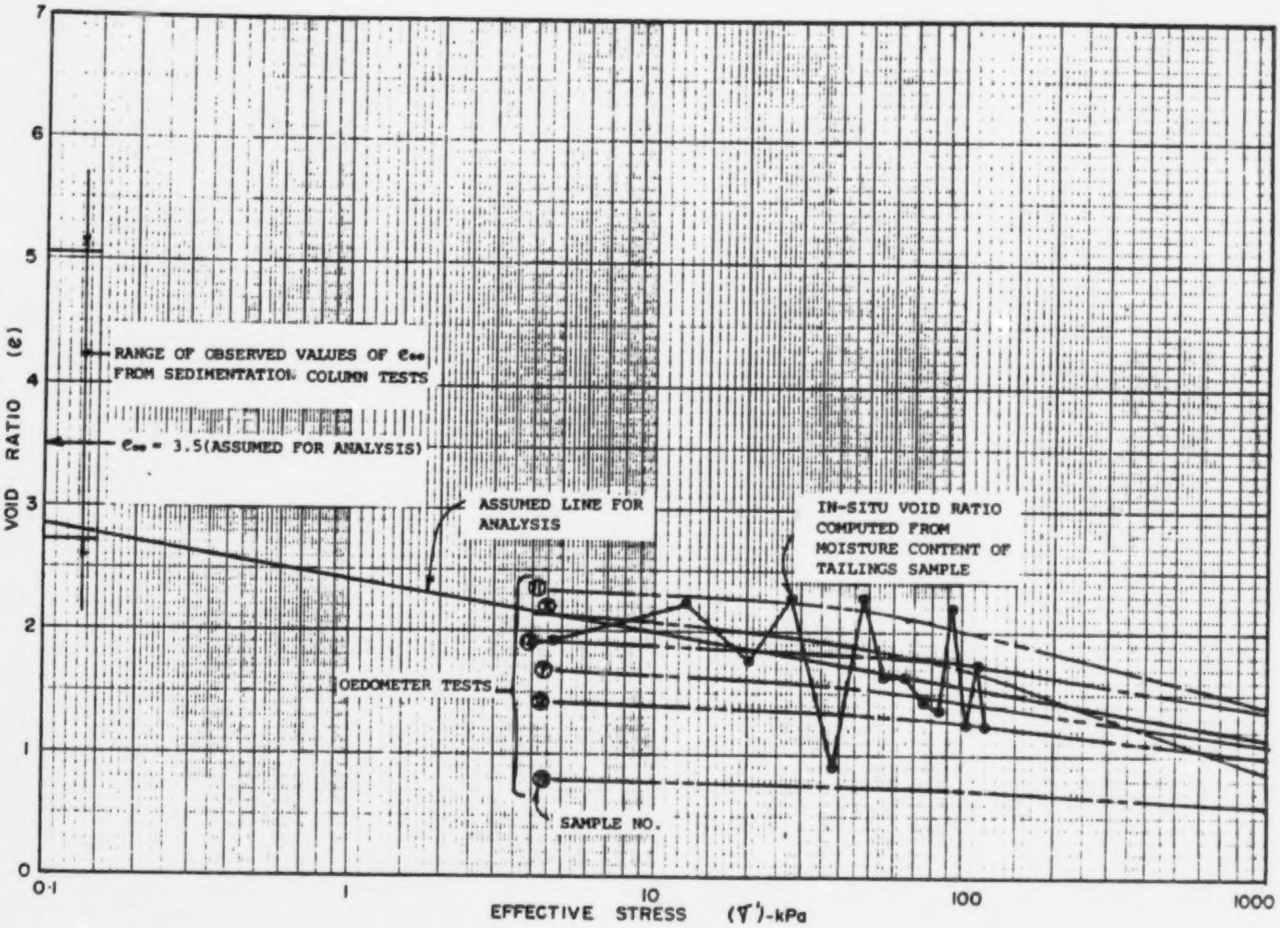


FIGURE 5 - 1B $e - \sigma'$ RELATIONSHIP
BOREHOLE NUTP - S2

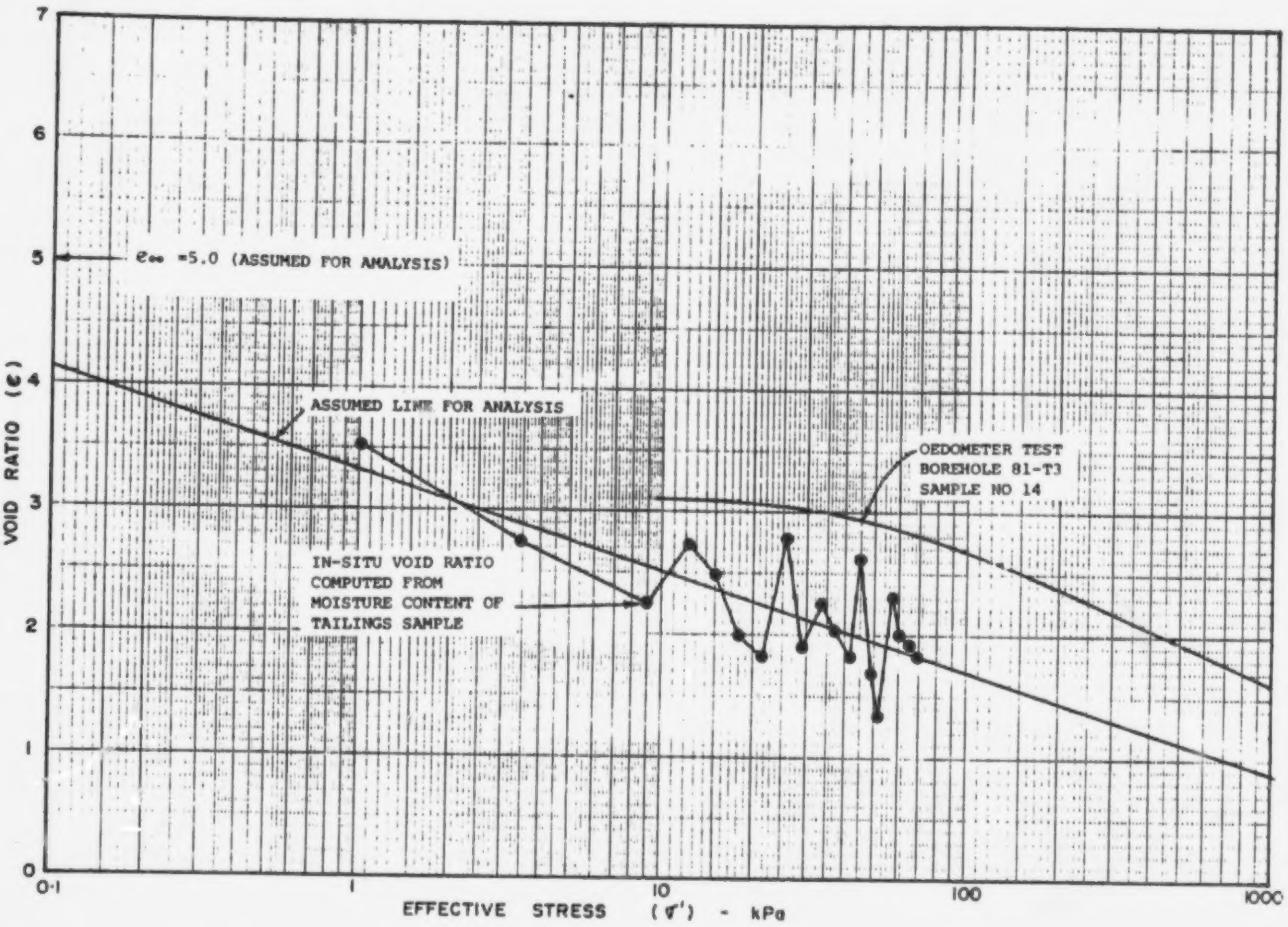


FIGURE 5 - 1C e - σ' RELATIONSHIP
BOREHOLE 81-T3

The range of the oedometer compressibility curves and the scattering of the in-situ void ratios about the mean, as seen in the figures, reflect the stratified and variable nature of the tailings deposit. Regression analyses of the in-situ void ratio data were carried out to obtain the idealized relationships shown in the figures, for input to the computer analyses. These best fit lines also appear to be in reasonable agreement with the average trends of the oedometer test data in the stress range above the preconsolidation pressures. This is consistent with the normally consolidated or slightly under consolidated nature of the tailings in place at the time of the field investigation.

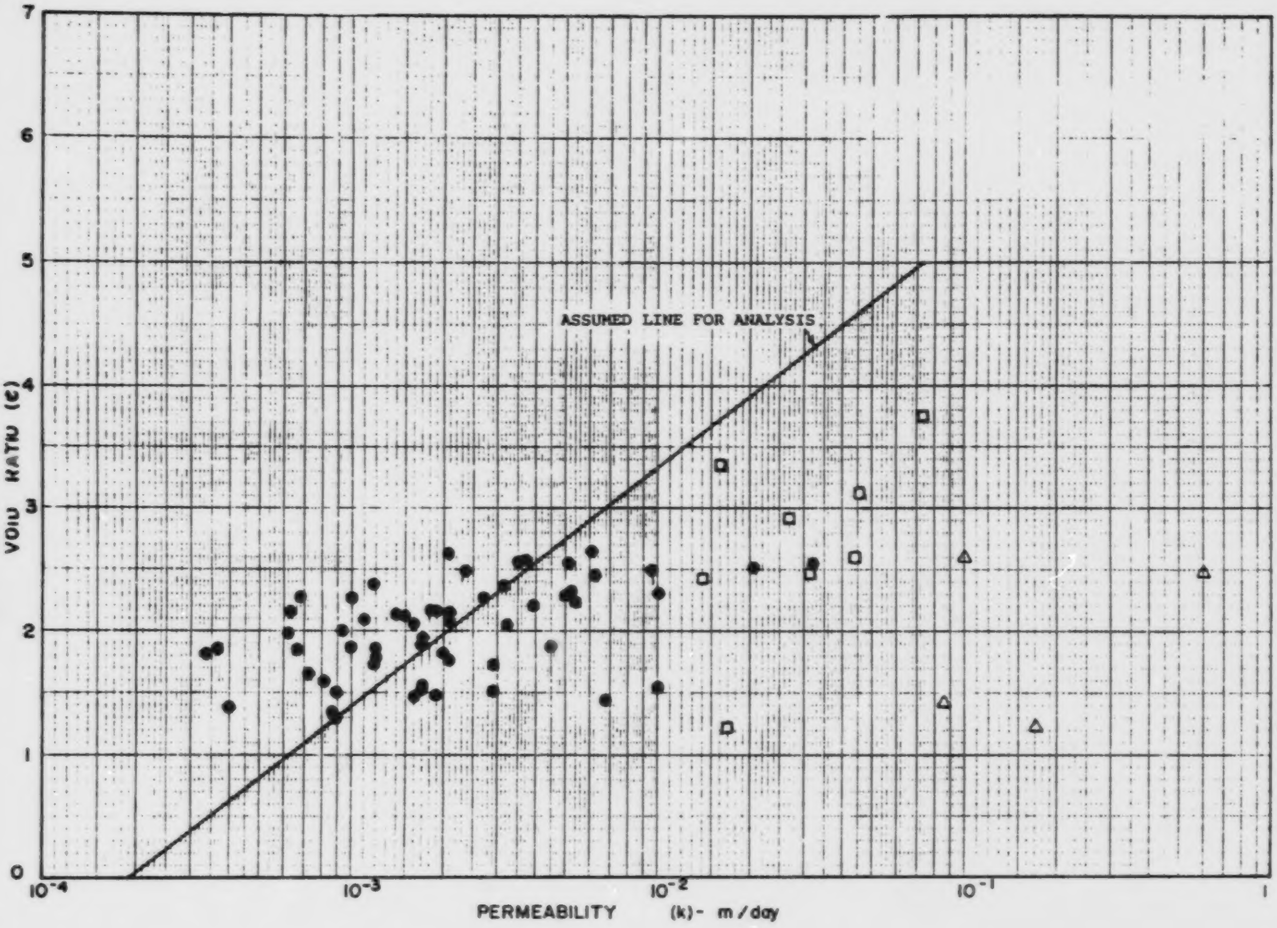
Initial Void Ratio

The initial void ratio varies throughout the depth of the deposit due to changes in grain size distribution of the tailings in place. An approximation is therefore involved in averaging data obtained from sedimentation column tests and from the in-situ void ratios determined near the surface of the tailings deposit.

The ranges of values and selected values for analyses are shown in Figures 5-1A, 5-1B and 5-1C.

Permeability

Summary plots of all the permeability test data obtained from laboratory and in-situ tests associated with Boreholes NUTP-S2, NUTP-S2 and 81-T3 are shown in Figures 5-2A, 5-2B and 5-2C. Because of the predominantly one-dimensional drainage in the vertical direction which is expected to take place due to consolidation of the tailings in the field, permeability in the vertical direction as determined in the oedometer tests was considered more appropriate for use in the analyses. Consequently, regression analyses were carried out to derive the best fit line through the oedometer test

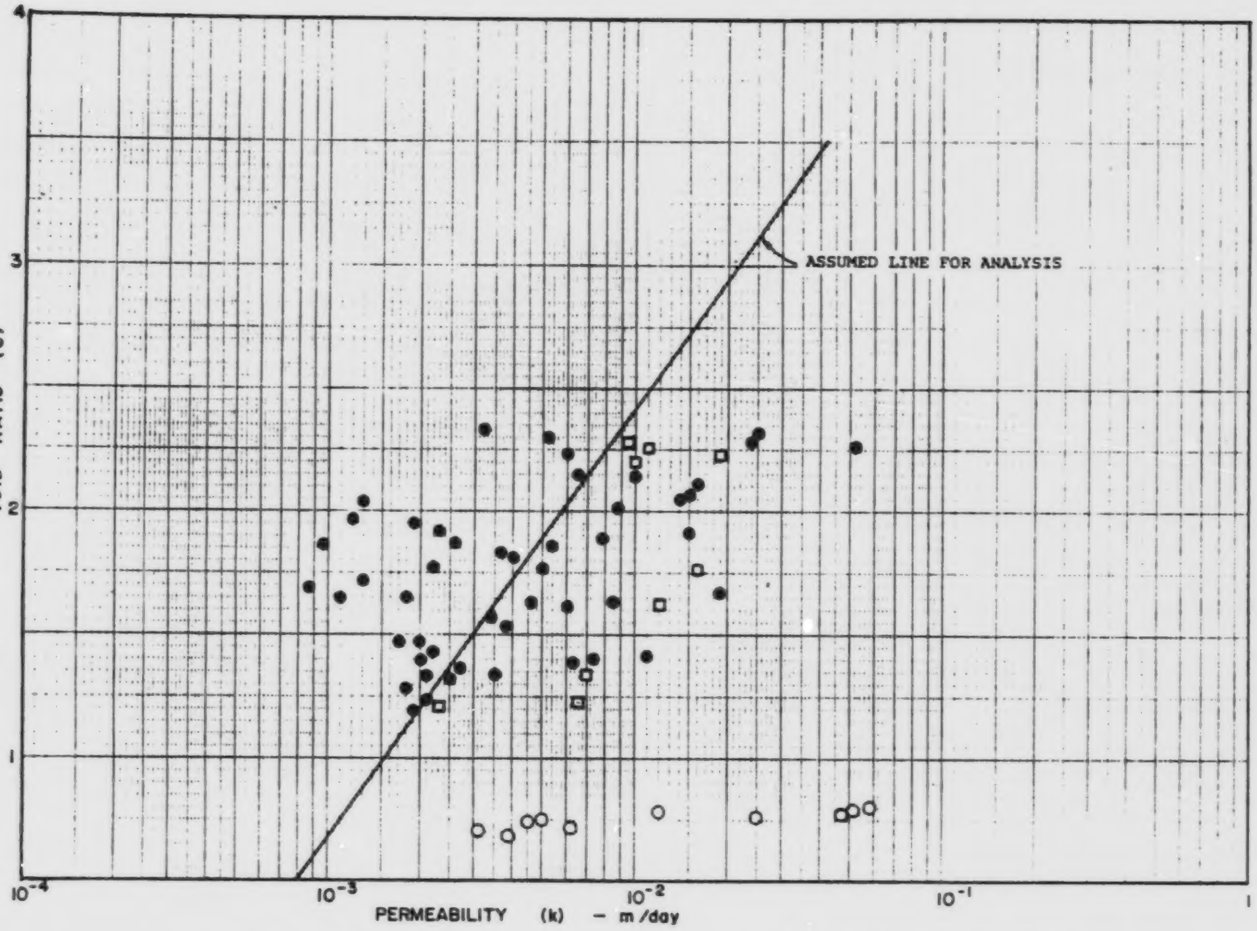


LEGEND

● --Oedometer Test Data
□ --Piezocone Test Data
△ --Field Falling Head Test Data

NOTE
Piezocone and Field Falling Head Test Data were not used in regression analysis to obtain the assumed line for analysis.

FIGURE 5 -2A e - k RELATIONSHIP
BOREHOLE NUTP - S3



LEGEND

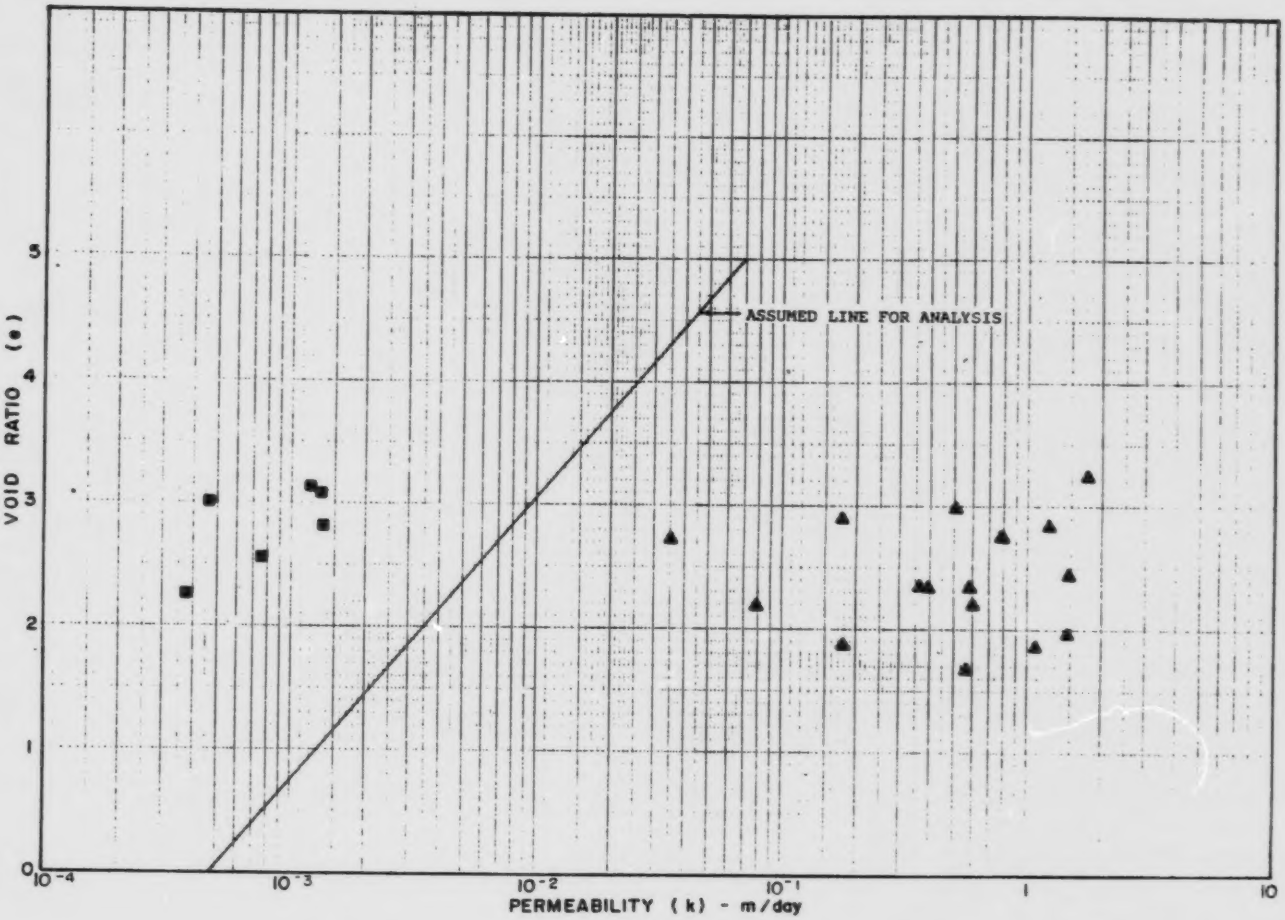
● -- Oedometer Test Data silty tailings

○ -- Oedometer Test Data sandy tailings

☐ -- Piezocone Data

NOTE
Oedometer Test Data of Sandy Tailings and Piezocone Data were not used in regression analysis to obtain the assumed line for analysis.

FIGURE 5 - 2B e - k RELATIONSHIP
BOREHOLE NUTP- S2



LEGEND

▲ -- FIELD FALLING HEAD TEST

■ -- OEDOMETER TEST
Borehole 81-T3 Sample 14

FIGURE 5 - 2C e - k RELATIONSHIP
BOREHOLE 81-T3

data points for use in consolidation modelling, in the case of Boreholes NUTP-S3 and NUTP-S2. The in-situ falling head tests and piezocone tests gave average permeability values representing the geometric mean of the horizontal and vertical permeabilities. Because of near horizontal stratification of the tailings, its horizontal permeability is anticipated to be considerably higher than its vertical permeability, probably by a factor of 2 to 4. The average permeability values obtained from these in-situ tests as shown in Figures 5-2A and 5-2B, which are higher than the mean line through the oedometer test data points, are therefore consistent with the anticipated trend.

In the case of Borehole 81-T3 the permeability data from a single oedometer testing was insufficient to provide a representative average e-k relationship for the full depth of the tailings. A certain amount of discretion was used therefore in selecting an idealized line for analyses as shown in Figure 5-2C.

Accretion Rate

In the analyses presented in Part A of the study, the accretion rates of the tailings in terms of Lagrangian height were assumed to be constant. This assumption was acceptable for that part of the study in view of the preliminary, illustrative nature of the analyses. The actual rate of accretion at any point in plan is of course not constant in reality, as illustrated by the tailings surface profiles shown in Figures 2-4 and 2-5. Close scrutiny of these profiles reveals two forms of variation of the accretion rate:

1) eratic variation due to changes in discharge point and/or the limits of the clarification pond, and

2) systematic variation related to the geometry of the impoundment basin.

The first form of variation does not have significant impact on the overall progress of consolidation and cannot be simulated accurately in any event. Therefore, no attempt is made in accounting for such details. The second form of variation, however, does result in well defined general trends of progressively decreasing rate of accretion with time as a result of increasing surface area of the impoundment facility as the level of the tailings rises. Therefore, it was considered necessary to refine the input data in this regard for this part of the study. For this purpose, the time plots of the observed heights of the tailings surface (convective height) at the borehole locations as shown in Figures 5-3A, 5-3B and 5-3C were used for guidance in developing the functional relationship between Lagrangian height of the tailings and time. Regression analyses were carried out to fit a function in the form

$$H_c = at^b$$

through the data points of observed convective heights, in which "H_c" is the convective height of the tailings surface at time "t" and "a" and "b" are empirical constants. Using the computed value of the exponent "b", a similar curve was then fitted between the origin and the point representing the maximum Lagrangian rates of accretion for input to the analyses. The maximum Lagrangian heights of the tailings at Boreholes NUTP-S2 and NUTP-S3 were computed by integrating the accumulated mass at the borehole location from unit weight information and the convective height of the tailings at the end of tailings deposition just before these boreholes were put down. The accumulated mass of tailings at Borehole 81-T3 at the end of deposition is unknown because the borehole was put down some five years before that time. The maximum Lagrangian height at this location was therefore estimated by extrapolating from the time of the boring to the end of tailings deposition.

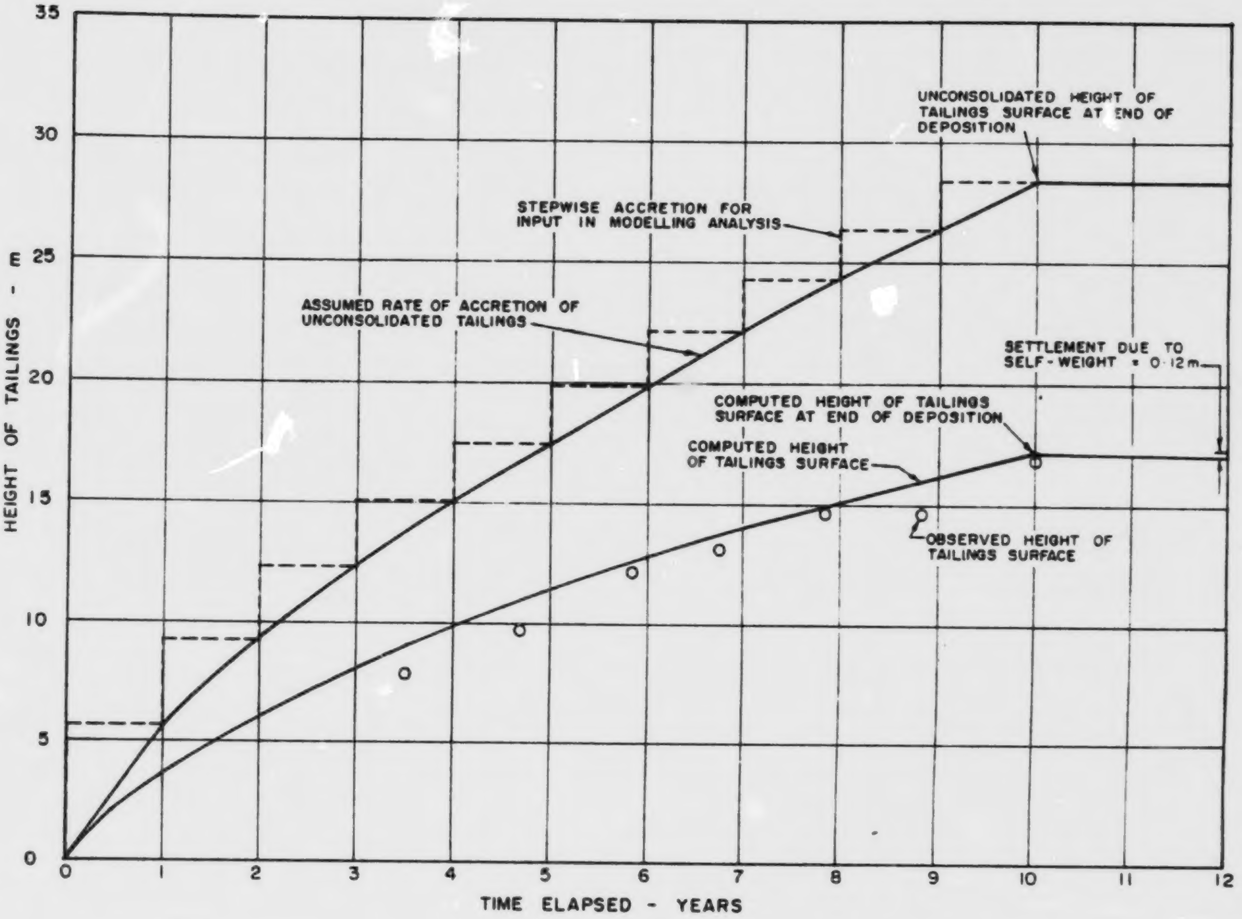


FIGURE 5 - 3A ACCRETION AND SELF - WEIGHT CONSOLIDATION CLARIFICATION POND AREA (BOREHOLE NUTP - S3)

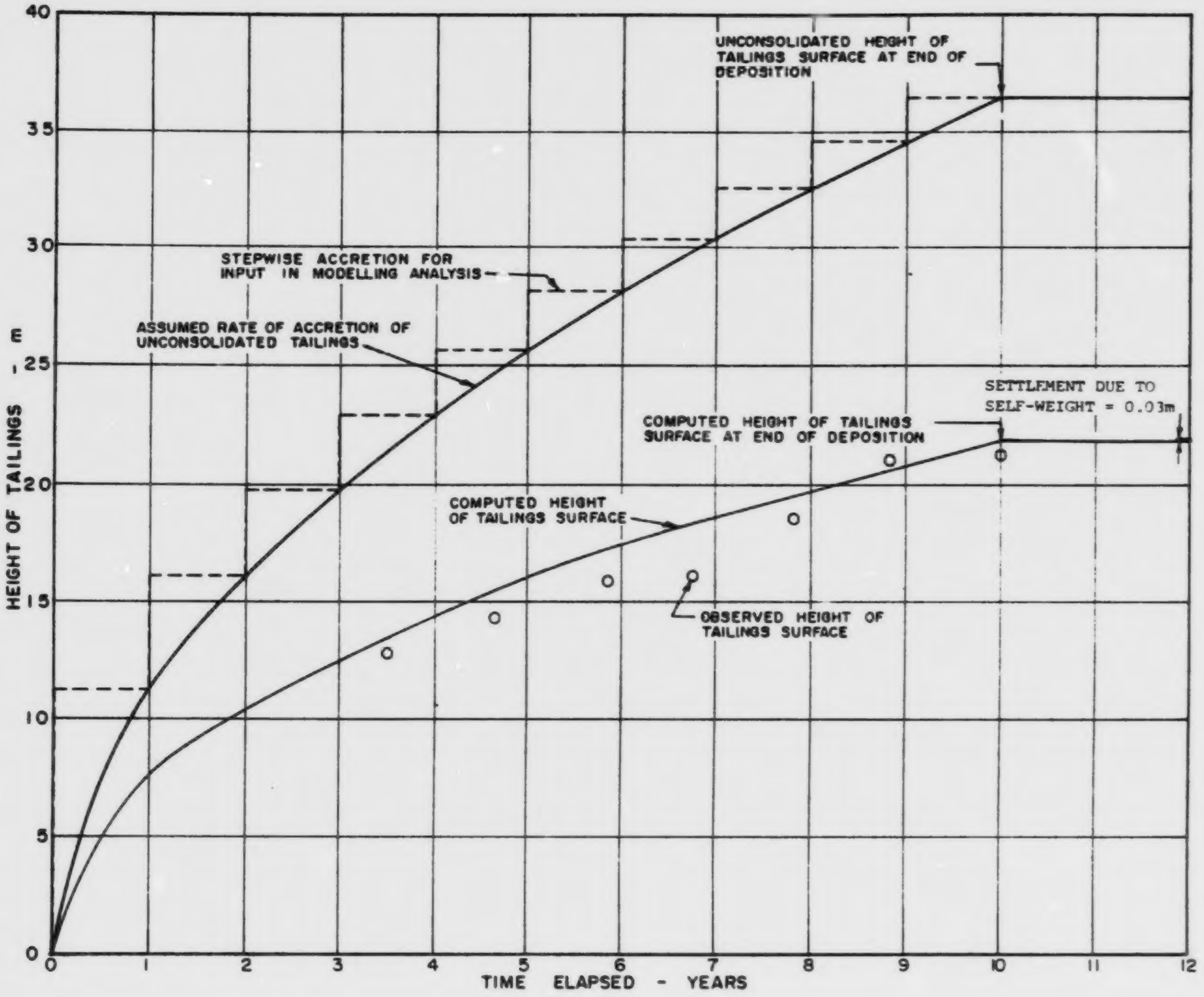


FIGURE 5 - 3B ACCRETION AND SELF-WEIGHT CONSOLIDATION SOUTH DAM AREA (BOREHOLE NUTP - S2)

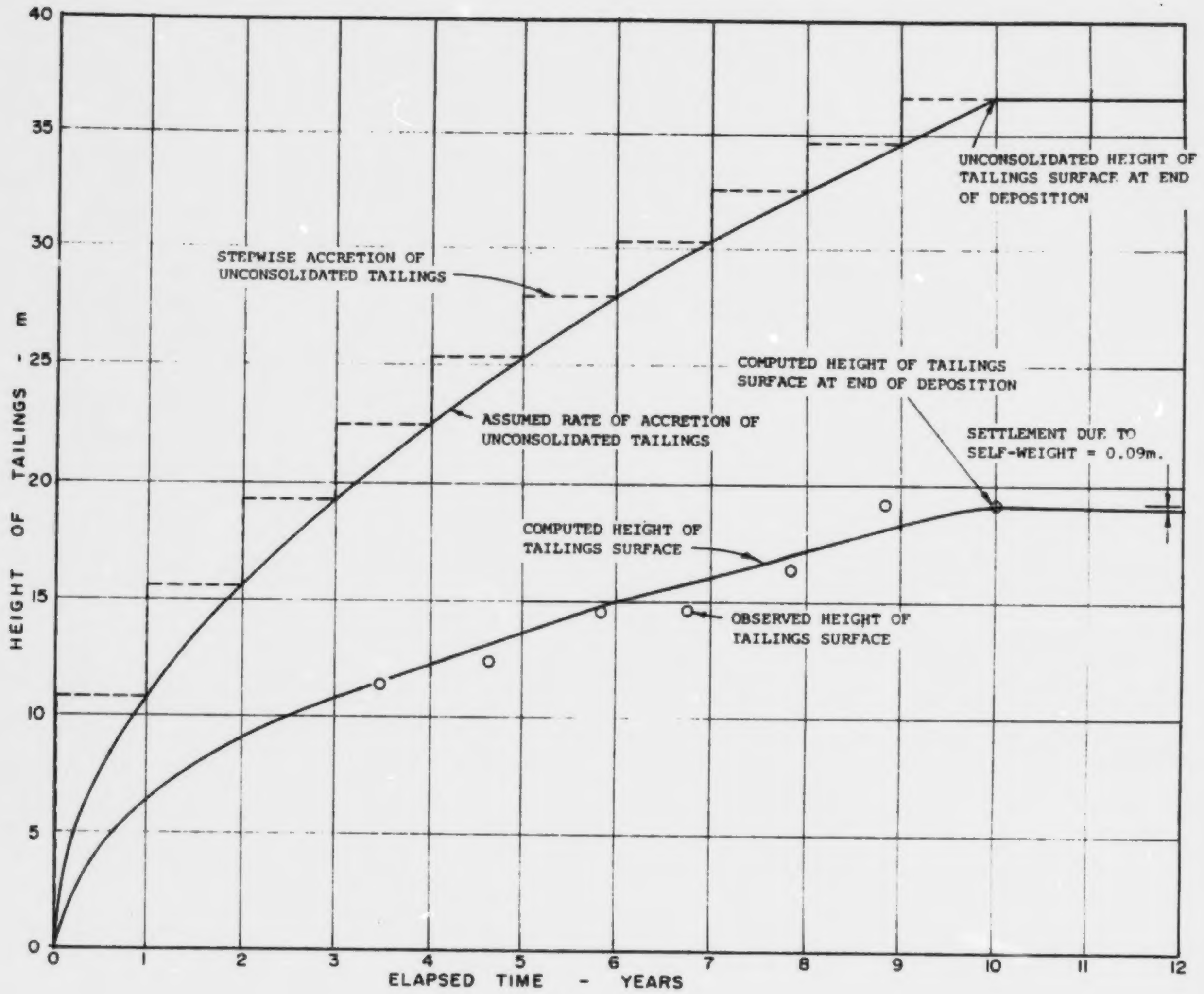


FIGURE 5 - 3C ACCRETION AND SELF-WEIGHT CONSOLIDATION SOUTH EDGE OF CLARIFICATION POND AREA (BOREHOLE 81 - T3)

5.2.2 Results of Analysis

Using the above input parameters, the self-weight consolidation during deposition at the location of Boreholes NUTP-S3, NUTP-S2 and 81-T3 was simulated by applying the computer program PCDDF in the manner described in the report on Part A of this study. The self-weight consolidation under submerged condition was assumed to extend over a two-year period after the end of deposition. The computed convective heights of the tailings surface at various times during deposition and for the two year period thereafter are shown in Figures 5-3A, 5-3B and 5-3C. Comparing the computed heights to those observed, it was considered that sufficiently close simulation of the consolidation behaviour of the tailings was achieved, thereby validating the model and input parameters. Good agreement was also obtained between the computed void ratio profile and the actual void ratio profile observed in the boreholes, as shown in Figures 5-4A, 5-4B and 5-4C.

The excess pore-water pressure distributions at the end of tailings deposition computed in the analyses are shown in Figures 5-5A, 5-5B and 5-5C. Piezometers installed during the investigation indicated essentially no excess pore-water pressures within the tailings. This observation therefore supports the results of the analysis.

Other computed values of interest are summarized below:

	NUTP-S3	NUTP-S2	81-T3
Percentage of primary condolidation settlement completed at end of tailings deposition	98.2	98.9	98.7
Additional self-weight consolidation settlements			
(a) two years after end of deposition (assumed beginning of decommissioning):	0.12 m	0.03 m	0.09 m
(b) ultimate:	0.21 m	0.16 m	0.23 m

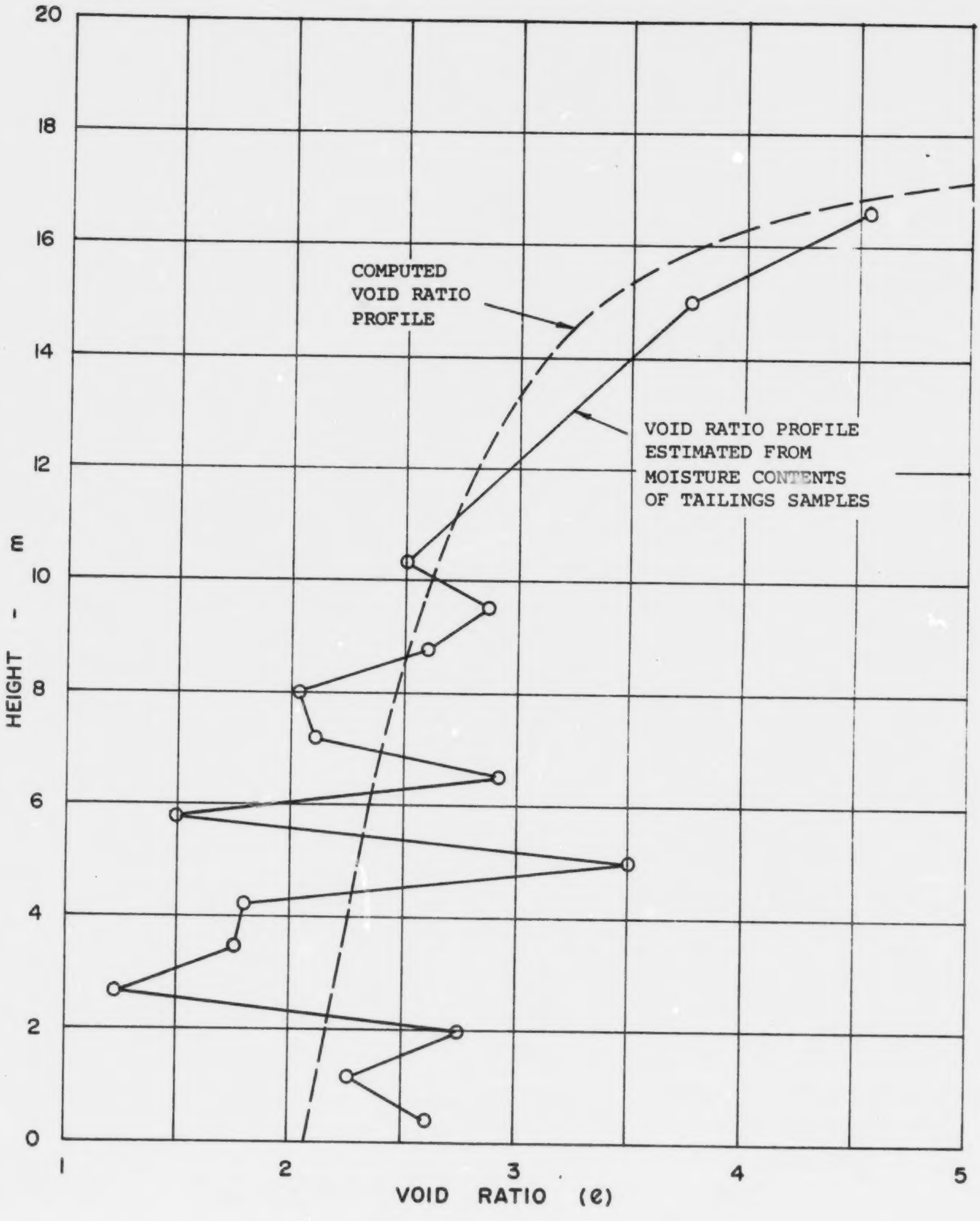


FIGURE 5 - 4A COMPARISON OF COMPUTED AND FIELD DETERMINED VOID RATIO PROFILES BOREHOLE NUTP- S3

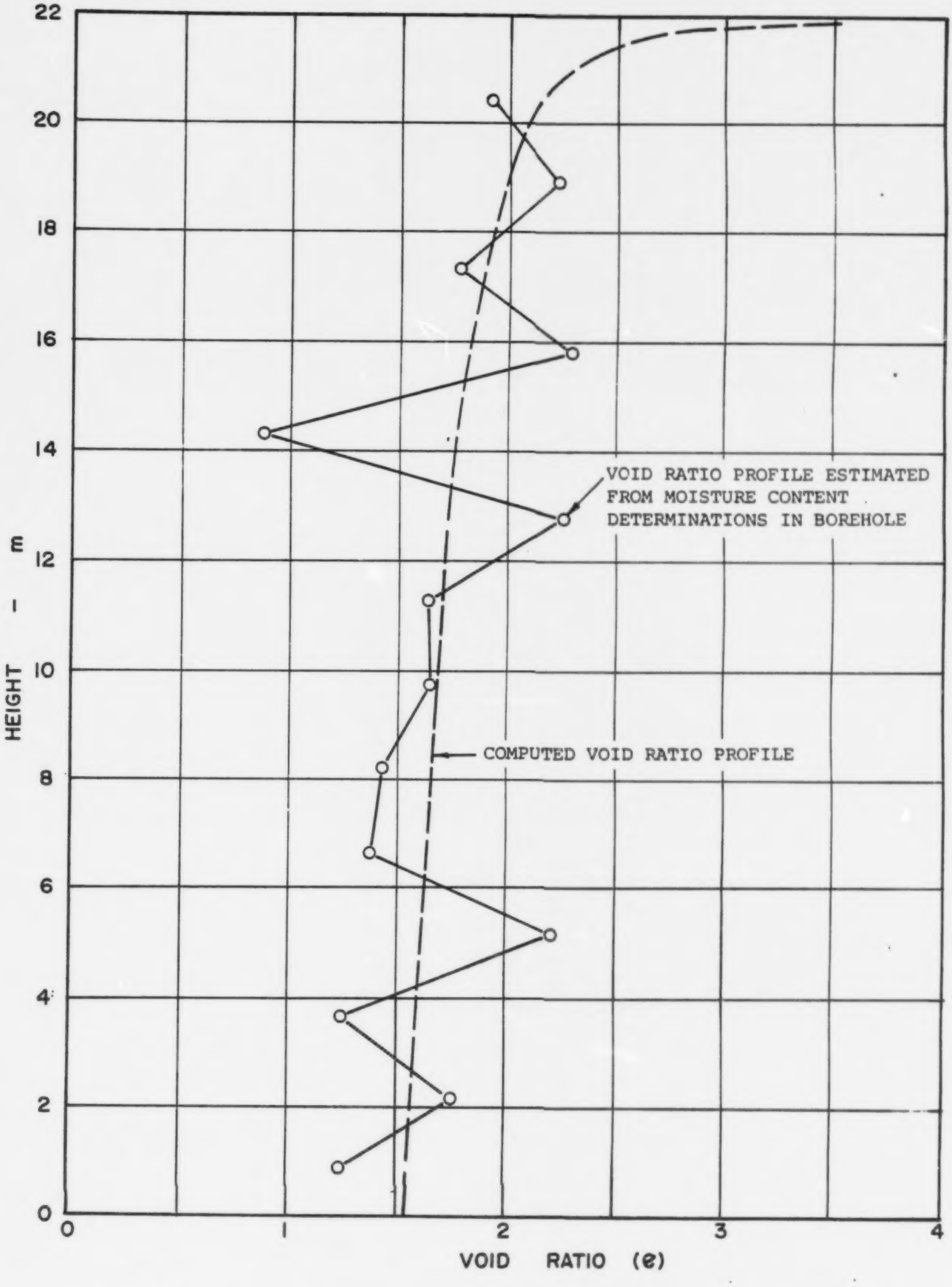


FIGURE 5 - 4B COMPARISON OF COMPUTED AND FIELD DETERMINED VOID RATIO PROFILES BOREHOLE NUTP - S2

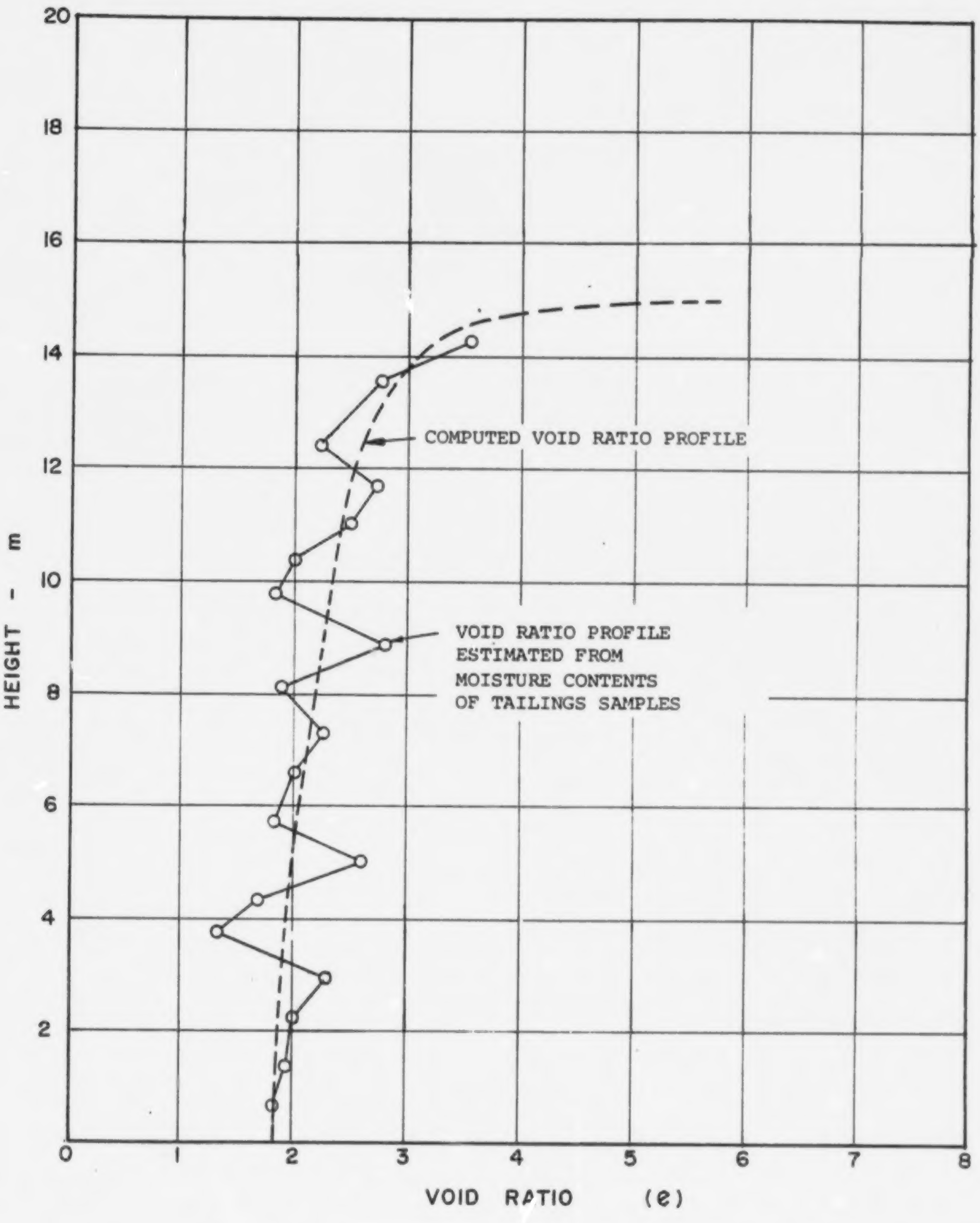


FIGURE 5 - 4C COMPARISON OF COMPUTED AND FIELD DETERMINED VOID RATIO PROFILES BOREHOLE 81 - T3

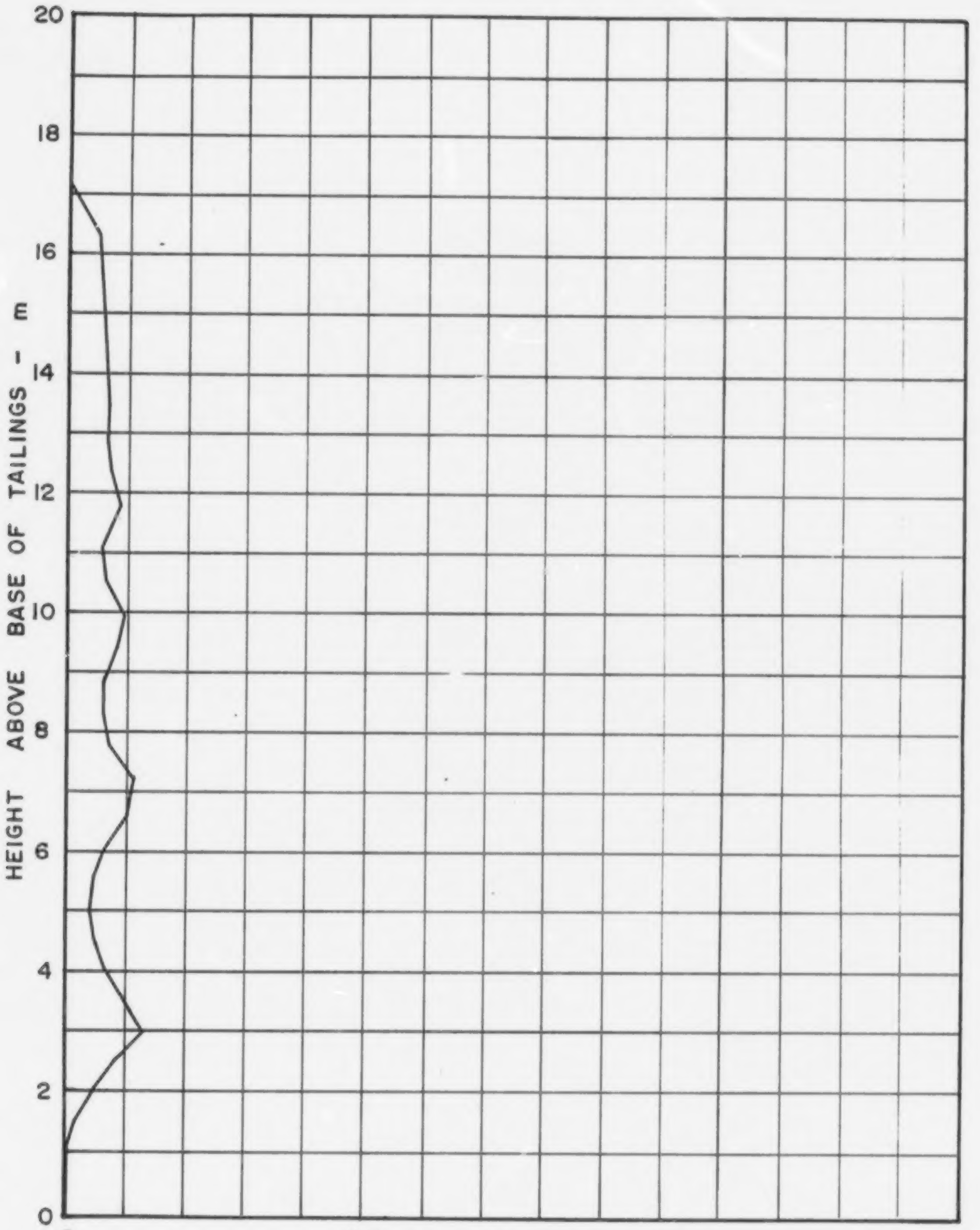


FIGURE 5 - 5A COMPUTED EXCESS POREWATER PRESSURE
BEGINNING OF DECOMMISSIONING
BOREHOLE NUTP - S3

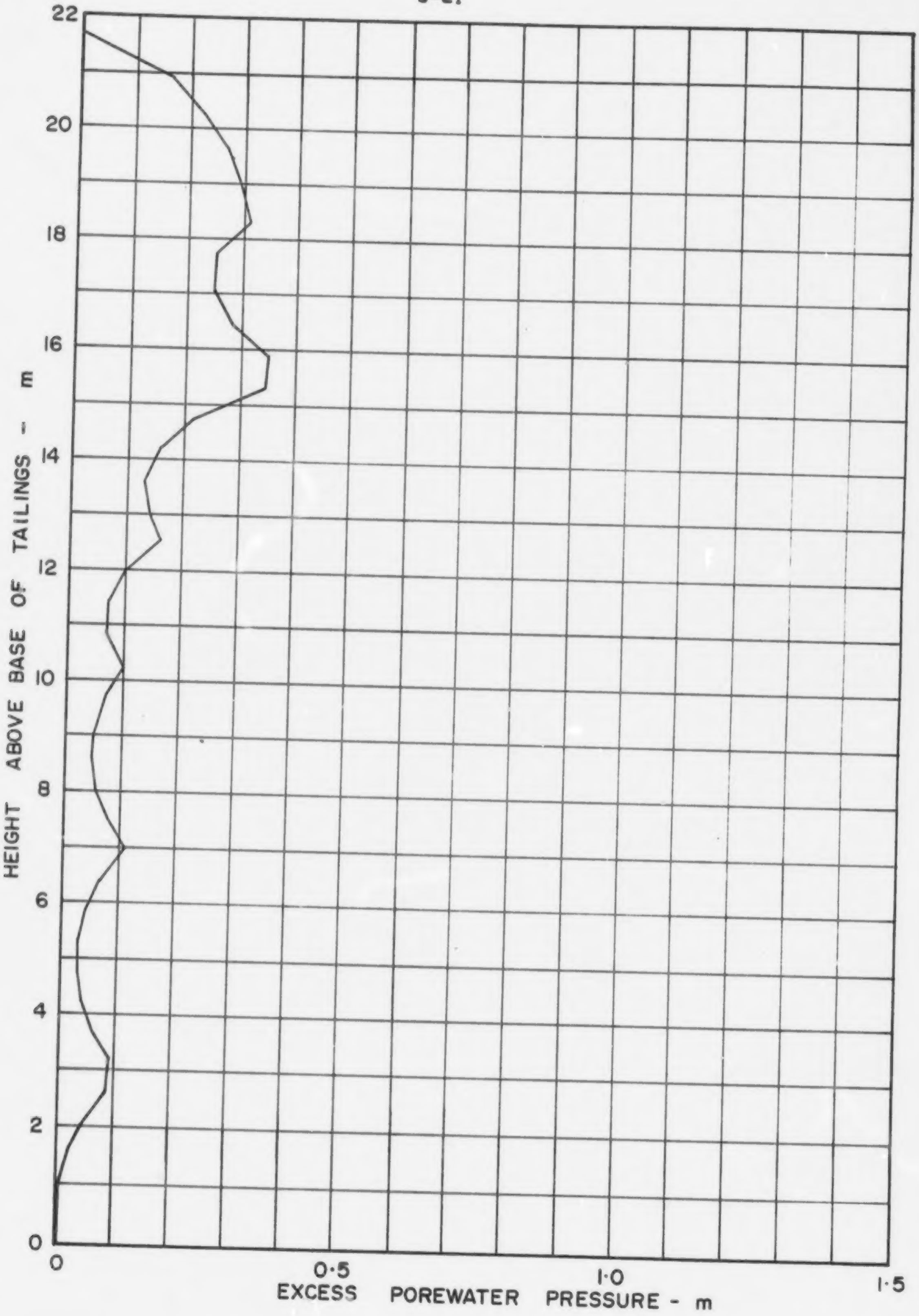


FIGURE 5 -5B COMPUTED EXCESS POREWATER PRESSURE
- BEGINNING OF DECOMMISSIONING
BOREHOLE NUTP - S2

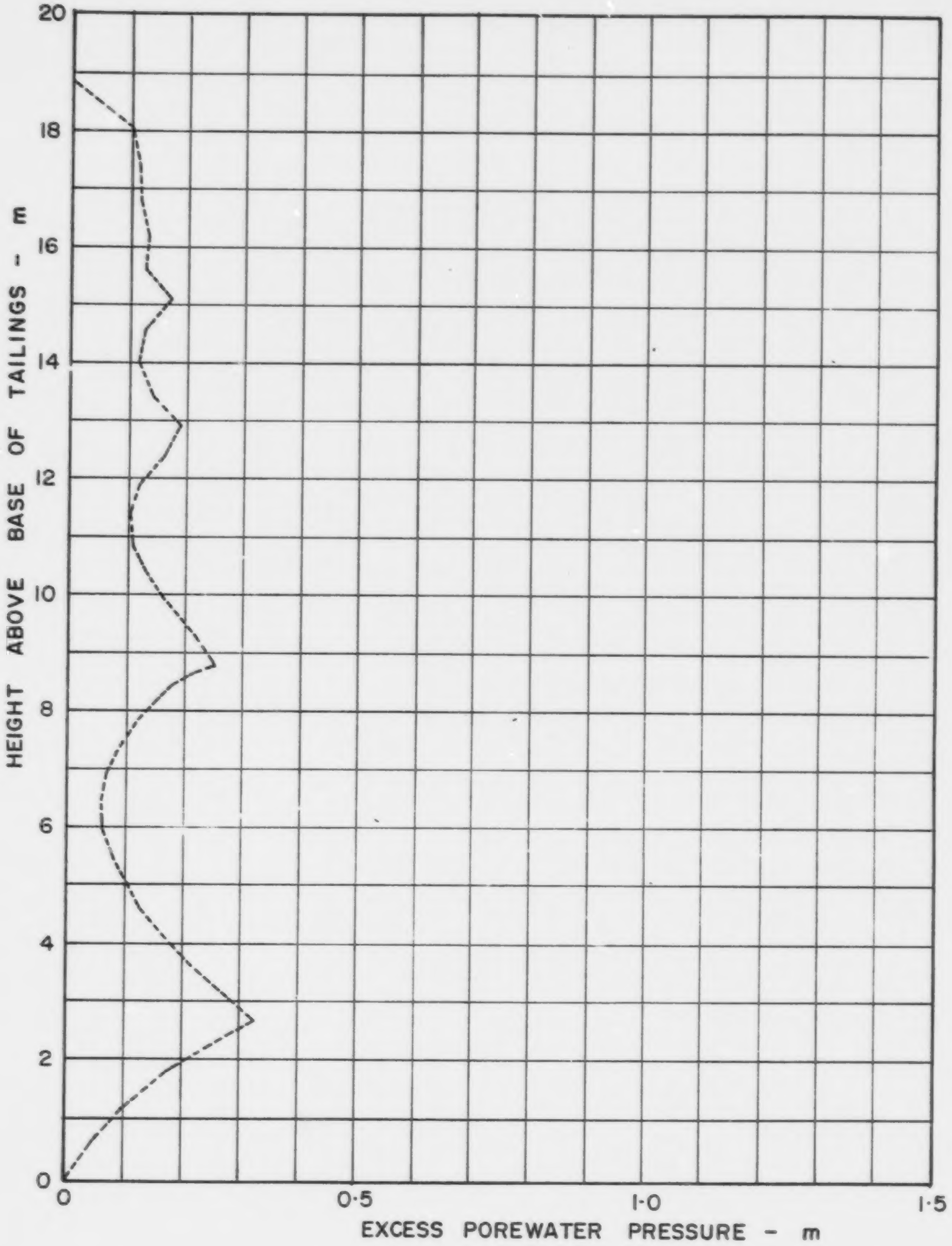


FIGURE 5 - 5C COMPUTED EXCESS POREWATER PRESSURE
BEGINNING OF DECOMMISSIONING
BOREHOLE 81 - T3

5.3 Settlement During Decommissioning

Analyses presented in Section 5.2 indicate that under fully submerged condition with the water level at or above the surface of the tailings, self-weight consolidation occurring during tailings deposition progressed at rates which enabled better than 98 percent of primary consolidation settlement to be completed at the end of the period. The residual settlements which can be anticipated to occur in the long term under such a fully submerged condition, will therefore be small, in the range of 0.09 m to 0.14 m as computed. The primary consolidation settlement occurs as a result of excursion of excess pore water without significant contribution from deformation of the soil particles under applied stress. This condition is fulfilled over the greater part of the consolidation process and the assumption of non-compressible soil particles in the theory of consolidation make it valid only for predicting primary settlement of fine grained soils. Secondary settlements "due to the gradual adjustment of the soil structure to stress, combined with the resistance offered by the viscosity of the adsorbed layers to a slippage between grains" (Terzaghi and Peck, 1948) are commonly observed towards the end of the primary consolidation process, resulting in continued settlement beyond the theoretically predicted end of primary consolidation. Such settlements are however, usually of a lower order of magnitude compared with primary settlements and can be neglected in the case under study.

Lowering of the water level anticipated to occur in the long term results in large desiccation shrinkage of the upper part of the tailings above the water level and additional consolidation of the underlying tailings resulting from increased effective stresses. The combined effect of these desiccation/consolidation processes is the occurrence of large surface settlements far greater than the residual settlements indicated above. To minimize the detrimental effects of such desiccation-induced settlements on the cover, it is evidently desirable to defer the construction of the cover by an appropriate period of time. To examine the time frame in such a scheme, it is naturally necessary to determine the settlement-time characteristics of the tailings deposit when exposed to desiccation under the site specific atmospheric environment. Modelling of the desiccation effects using the program PCDDF is described below.

5.3.1 Input Data

In addition to the input data given earlier in Section 5.2.1, information related to precipitation, runoff, evaporation and desiccation characteristics of the tailings, is required for this part of the analyses.

Average precipitation at the Collins Bay Station given in Table 2-1 and estimated average monthly evaporation shown in Table 2-2 both under Section 2.1.2, were used for input in the analyses. The use of the precipitation record, instead of rainfall, in the analyses allowed infiltration of snow melt to be taken into account. This approach is on the conservative side in estimating the effectiveness of desiccation in pre-inducing settlements prior to cover construction.

The relatively steep surface gradients of the tailings, averaging 1.5% with local areas up to 10%, within the clarification pond area and the greater part of the area south of it, as shown in Figures 2-4 and 2-5, combined with the fine-grained nature of the tailings, will ensure excellent surface drainage even without the special measures mentioned earlier. However, to allow for some ponding in shrinkage cracks formed in the tailings surface after initial desiccation, a conservatively assumed value of 0.8 for the runoff coefficient, C_D, was used in the analyses.

Based on the limited test work on the desiccation characteristics of the tailings, the limit void ratios assumed for input into the analyses were:

$SL = 2.50$
$DL = 2.15$

The maximum depth of second stage drying was assumed to be 2.0 m.

5.3.2 Results

The results of the analyses are shown in Figures 5-6A, 5-6B and 5-6C for the clarification pond, south beach and transition areas, respectively. These show that desiccation settlement will take place rapidly over the initial period after drainage of the clarification pond. Due to the more sandy nature of the tailings in the south beach area, with a lower initial void ratio profile at the beginning of the desiccation proceeds, the computed total settlement is relatively small, less than 0.5 m, and is predicted to be essentially completed over a period of two years (Figure 5-6B). The predicted magnitude and duration increase to about 1.3 m over 5 years in the transition area (Figure 5-6C) and about 1.9 m over 6 years in the clarification pond area (Figure 5-6A).

5.4 Cover Settlement

The analyses presented in Section 5.3 give approximate magnitudes and duration of desiccation induced settlements which could be anticipated to take place if the tailings is exposed to atmospheric evaporation. It must be realized of course that, notwithstanding the test work which has been carried out to enable input parameters to the analyses to be estimated with reasonable accuracy, variations in the atmospheric environment, surface drainage conditions and properties of the tailings from place to place are virtually impossible to predict or define precisely. It is therefore necessary in practice to monitor the actual behaviour of the tailings during the process of decommissioning to obtain information for reviewing the results of predictive modelling such as those described in Sections 5.2 and 5.3. With such a review, time scheduling for final placement of the cover can be adjusted with the view of optimizing the benefit of desiccation and allowing post-construction cover settlements to be predicted with a higher degree of accuracy and confidence.

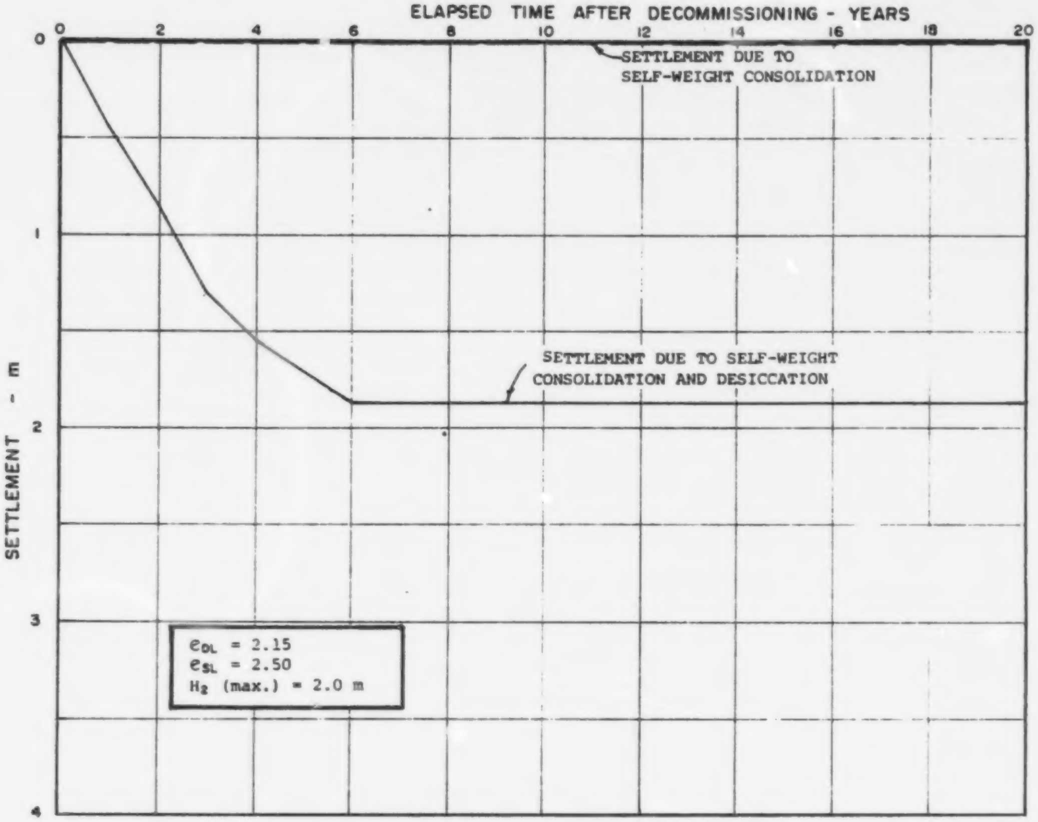


FIGURE 5 - 6A DESICCATION - INDUCED SETTLEMENT (NO COVER)
CLARIFICATION POND AREA (BOREHOLE NUTP - S3)

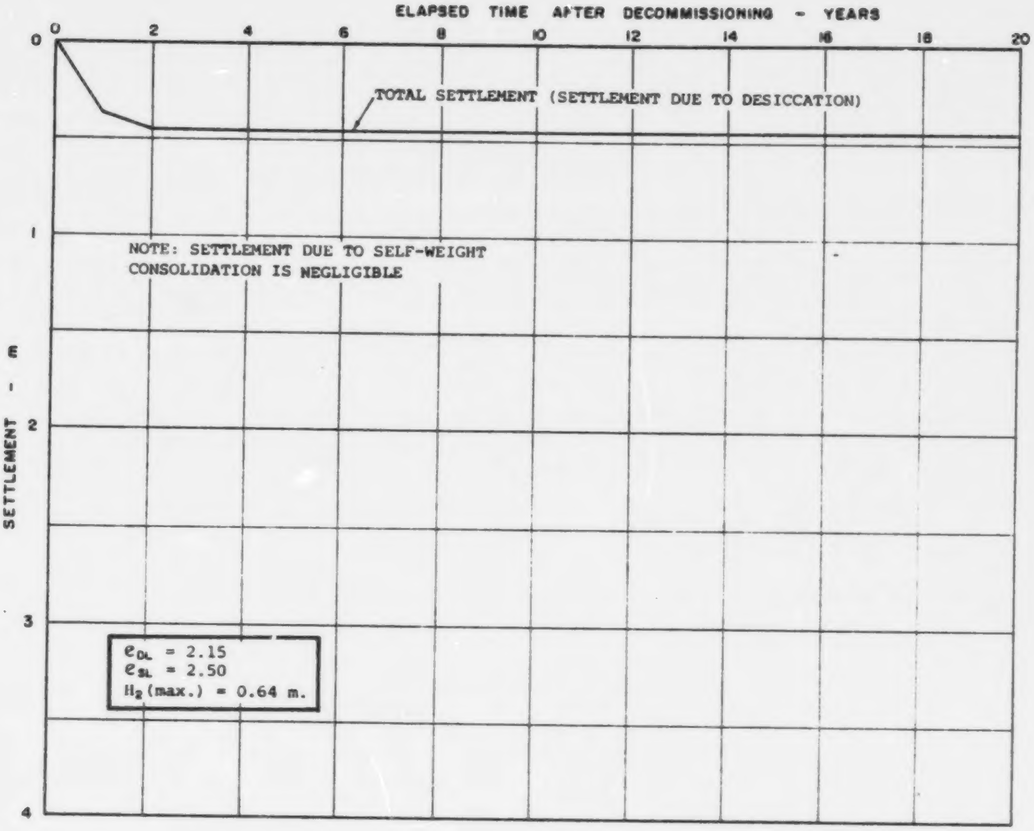


FIGURE 5-6B DESICCATION - INDUCED SETTLEMENT (NO COVER)
SOUTH BEACH AREA (BOREHOLE NUTP-S2)

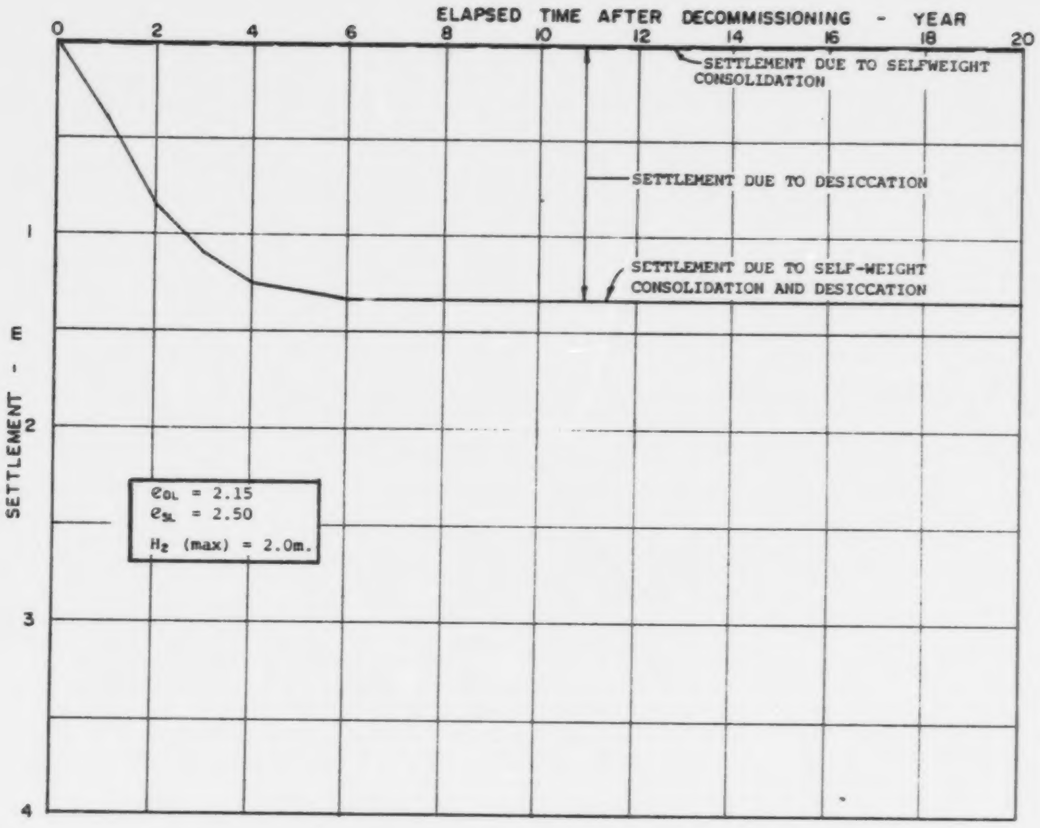


FIGURE 5 - 6C DESICCATION - INDUCED SETTLEMENT (NO COVER) TRANSITION AREA (BOREHOLE 81 - T3)

For illustrative purposes, cover settlements for two cases were analysed:

1) cover constructed immediately following drainage of the clarification pond, and
2) cover construction deferred by a period of 3 years after drainage of the clarification pond

The first case of course is not viable in practice. It was merely intended to provide comparison for illustrating the benefit realized by the deferred cover construction.

Because post-construction cover settlement is sensitive to the effectiveness of the cover as an evaporation barrier, and cover design is not within the scope of this study, it was necessary to consider the upper and lower bound cases in the analyses in which the cover was assumed to be either entirely ineffective or fully effective in preventing evaporation from the underlying tailings.

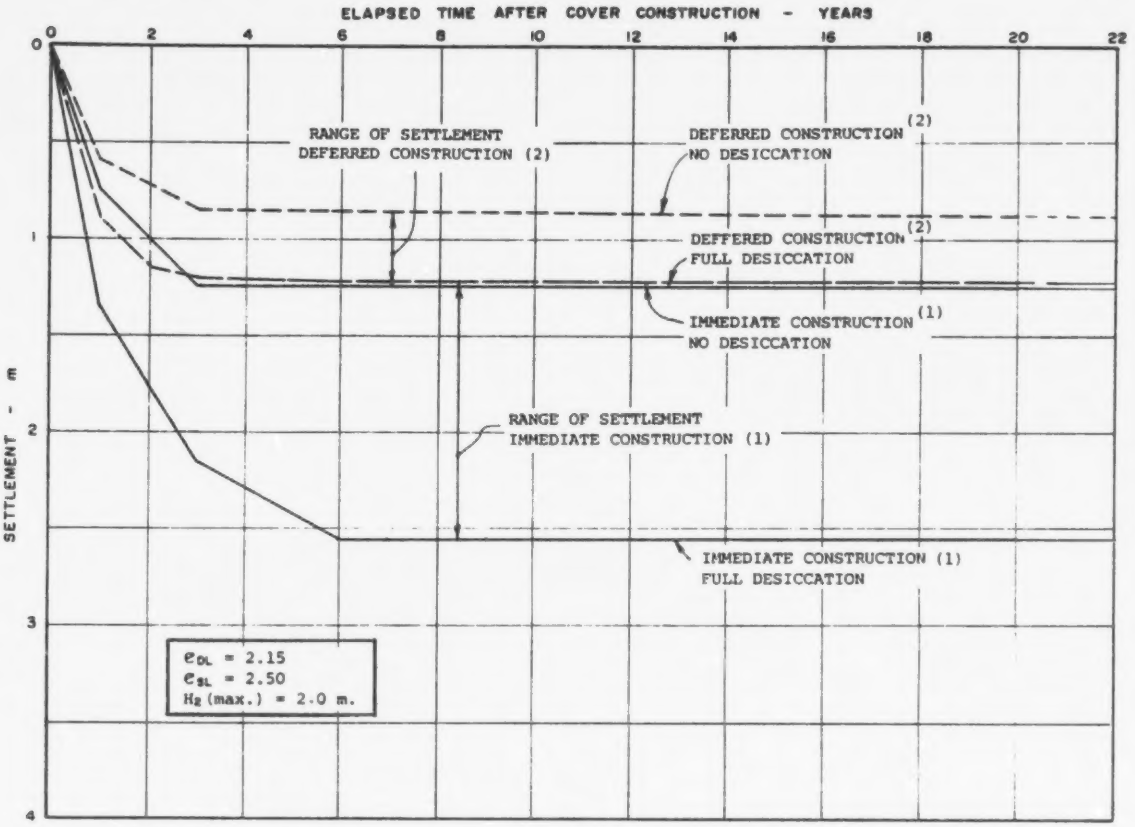
5.4.1 Input Data

The surcharging effect of a 0.6 m thick earth/rock fill cover was simulated by an equivalent layer of tailings of equal weight, using the procedure as outlined in Section 4.4 of the report on Part A of this study. The unit weight of the cover was taken to be 19.6 kN/m^3.

To simulate the case where the cover is fully effective in preventing evaporation from the tailings, monthly pan evaporation rates were input as zero year round.

5.4.2 Results

The results of the analyses are shown in Figures 5-7A, 5-7B and 5-7C on the following pages. From these results, it can be seen that substantial

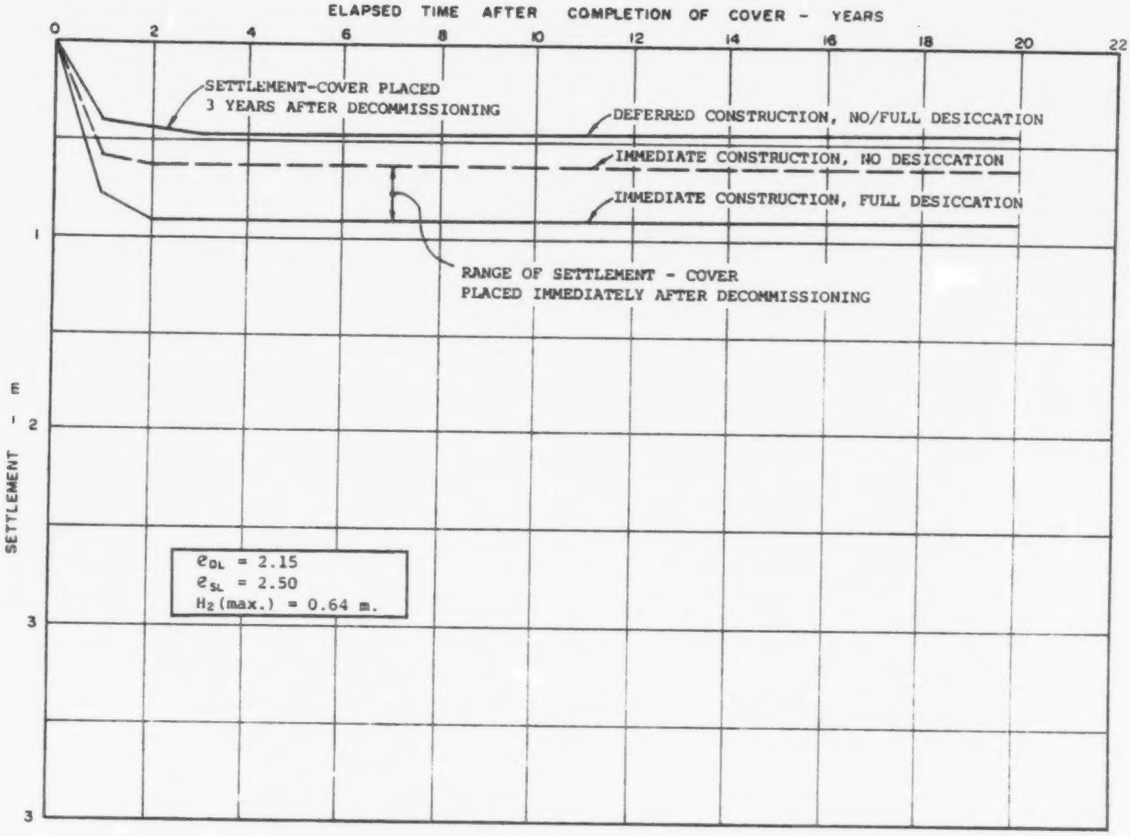


NOTES:

1. "IMMEDIATE CONSTRUCTION" REFERS TO THE CASE IN WHICH COVER IS PLACED IMMEDIATELY FOLLOWING DRAINAGE OF THE CLARIFICATION POND

2. "DEFERRED CONSTRUCTION" REFERS TO THE CASE IN WHICH COVER PLACEMENT IS DEFERRED BY 3 YEARS AFTER DRAINAGE OF THE CLARIFICATION POND.

FIGURE 5-7A COVER SETTLEMENT
CLARIFICATION POND AREA
(BOREHOLE NUTP - S3)

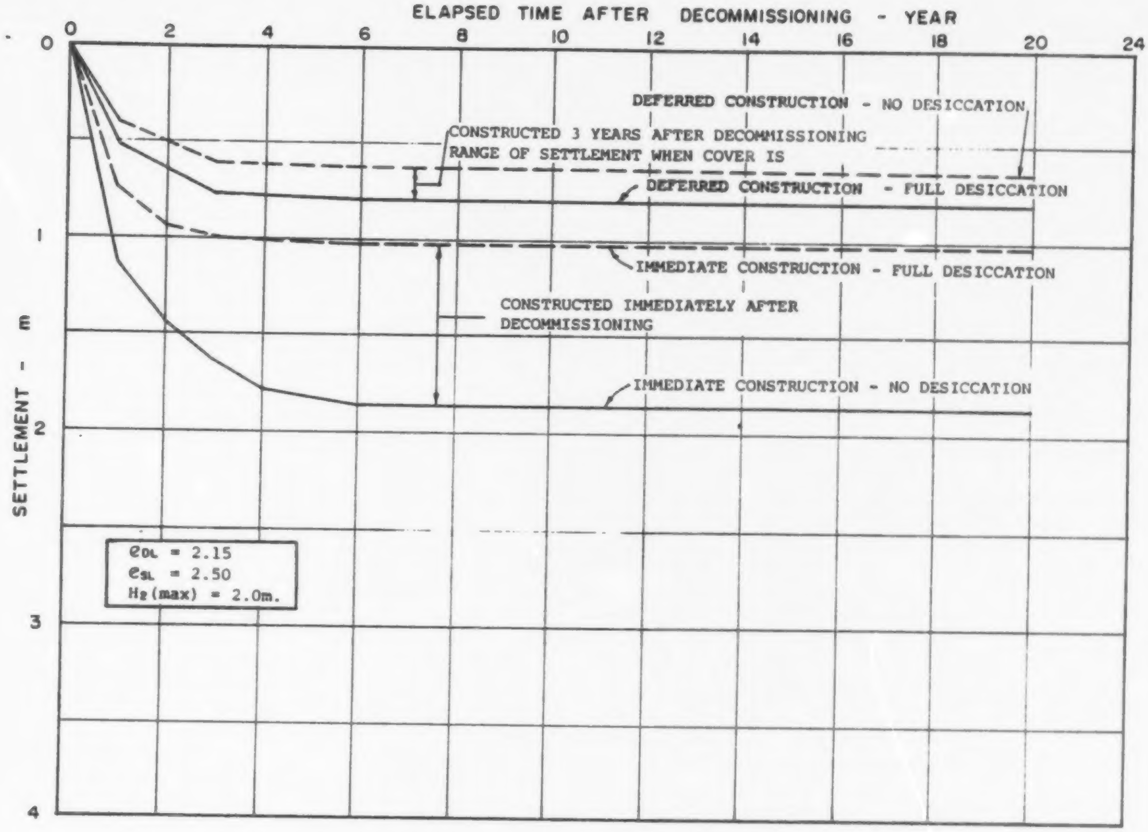


NOTES:

1. "IMMEDIATE CONSTRUCTION" REFERS TO THE CASE IN WHICH COVER IS PLACED IMMEDIATELY FOLLOWING DRAINAGE OF THE CLARIFICATION POND

2. "DEFERRED CONSTRUCTION" REFERS TO THE CASE IN WHICH COVER PLACEMENT IS DEFERRED BY 3 YEARS AFTER DRAINAGE OF THE CLARIFICATION POND.

FIGURE 5-7B COVER SETTLEMENT
SOUTH BEACH AREA
(BOREHOLE NUTP-S2)



NOTES:

1. "IMMEDIATE CONSTRUCTION" REFERS TO THE CASE IN WHICH COVER IS PLACED IMMEDIATELY FOLLOWING DRAINAGE OF THE CLARIFICATION POND

2. "DEFERRED CONSTRUCTION" REFERS TO THE CASE IN WHICH COVER PLACEMENT IS DEFERRED BY 3 YEARS AFTER DRAINAGE OF THE CLARIFICATION POND.

FIGURE 5-7C COVER SETTLEMENT
TRANSITION AREA
(BOREHOLE 81-T3)

reduction in the long term settlements could be realized by deferring the cover construction. With the three year deferment of construction, the cover settlement in the clarification pond area (Figure 5-7A) is reduced to a range of about 0.8 m to 1.2 m. The bulk of the settlement will occur within the first two years after cover construction.

As anticipated, computed settlements decrease in the direction towards the South Dam. The computed range of settlements in the transition area (Figure 5-7C) with deferred construction is 0.6 m to 1.0 m and that in the south beach area (Figure 5-7B) is 0.5 m. In the latter case, the desiccation process is essentially completed prior to cover construction and therefore the post-construction settlement is almost entirely due to the weight of the cover and is practically independent of evaporative effects.

5.5. Effect of Thawing - North Beach Area

The hypothetical scenario of the frozen tailings within the north beach area thawing out completely sometime in the future is examined herein. The possibility of this event occurring is believed to be extremely remote, and if it occurred at all, would be due to a global warming trend which may take place over a very long time frame at very slow rates. Consequently, any thawing of the tailings in that event would also occur at extremely slow rates. A further lag in time is also involved due to the time taken in the consolidation of the thawed tailings, before surface settlement occurs from this effect. As mentioned before, it is not possible to predict the actual rates of thawing of the tailings and of the surface settlement taking place as a result.

An attempt was made, however, to estimate the order of magnitude of ultimate settlement which could take place if the tailings thawed out completely. This was carried out by computing the ultimate void ratio profiles as shown in Figure 5-8, using two cases of assumed average compressibility characteristics for the tailings as follows:

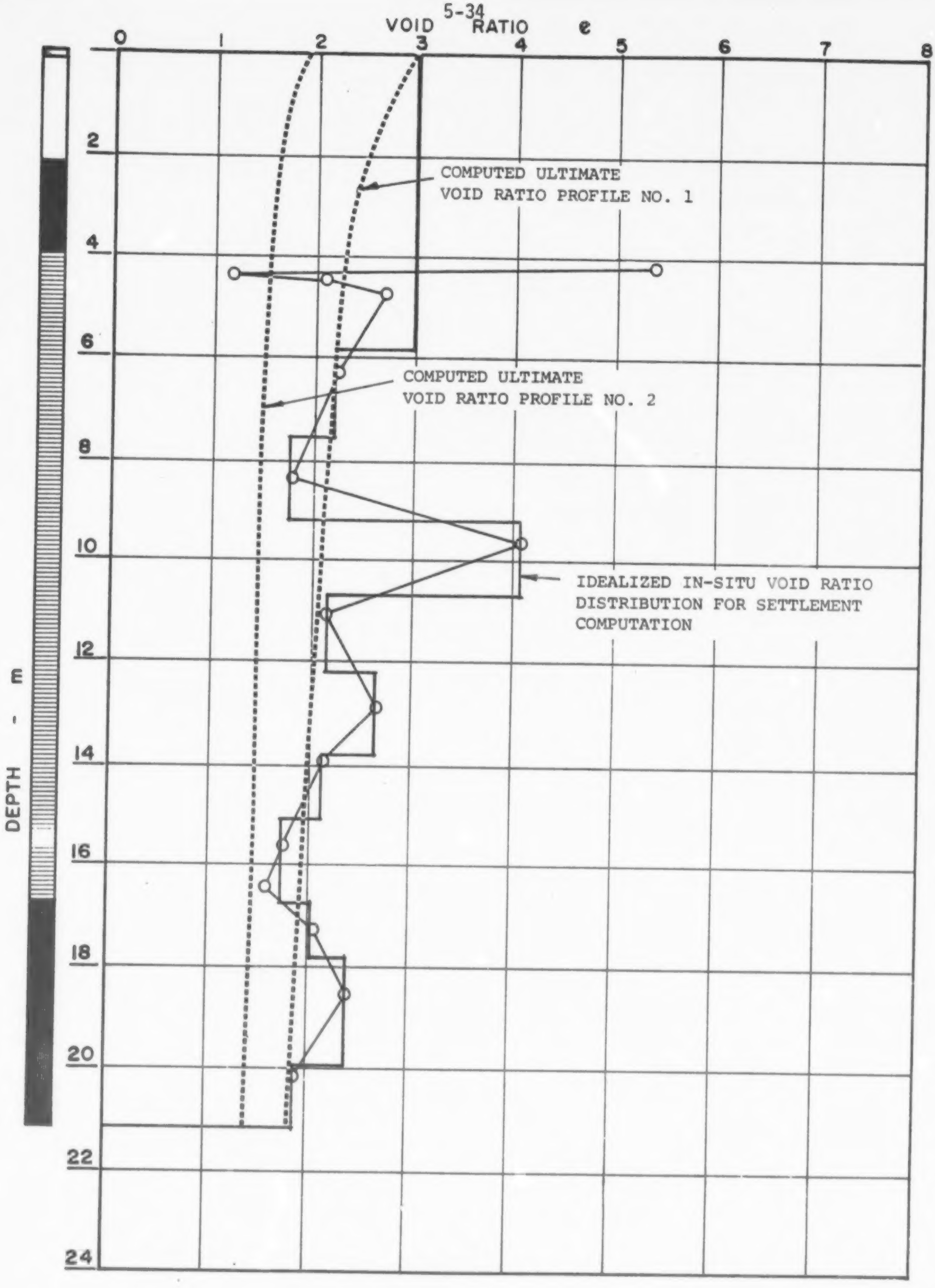


FIGURE 5-8 VOID RATIO PROFILES FOR SETTLEMENT COMPUTATION NORTH BEACH AREA (BOREHOLE S-1).

Case 1 $e_o = 2.90$
$C_c = 0.50$

Case 2 $e_o = 1.96$
$C_c = 0.26$

The in-situ void ratio profile of the tailings under existing frozen conditions as determined from sample moisture contents in Borehole NUTP-S1 is also shown in Figure 5-8 together with an idealized profile for computation purposes. By integrating the volume changes between the idealized, existing void ratio profile and the ultimate void ratio profiles, the computed ultimate settlements if completely thawed, are

Case 1: 2.15 m
Case 2: 5.40 m

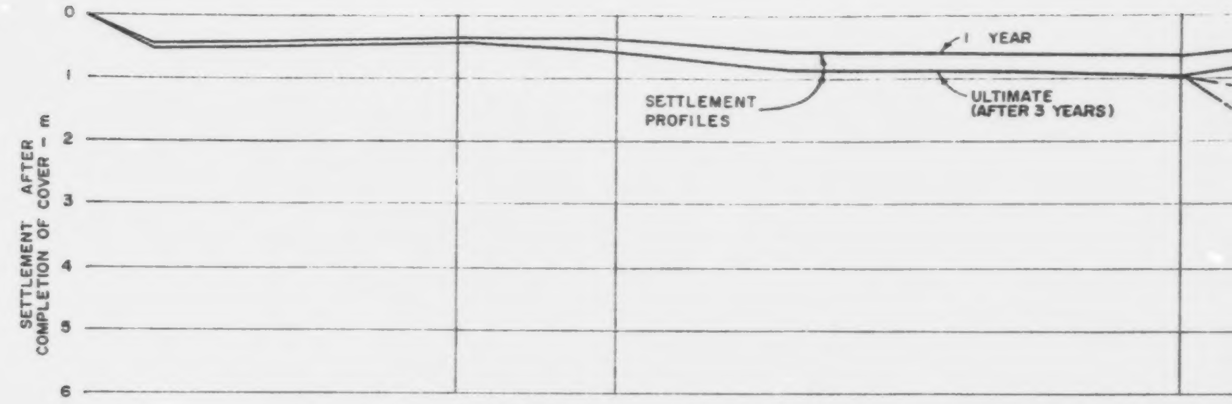
It must be noted again that the scenario of complete thawing of the tailings is hypothetical and is included only to illustrate the extreme worst case if a radical change in local climate to considerably higher temperatures takes place. The northern location of the site in an area of known sporadic permafrost occurrence and low mean annual temperatures provides good assurance that the frozen tailings should remain frozen under current conditions. The very fact that the bulk of the tailings in this area has remained frozen to date after a period of some ten years of continuous deposition, is good evidence to the likelihood of the frozen tailings to remain frozen in the long term. This is because the heat input during the period of deposition is greater than in the long term due to the heat contained by the fresh tailings from the mill and shorter heat conduction pathways from the surface in the summmer seasons comparted to the final conditions.

5.6 Settlement Profile

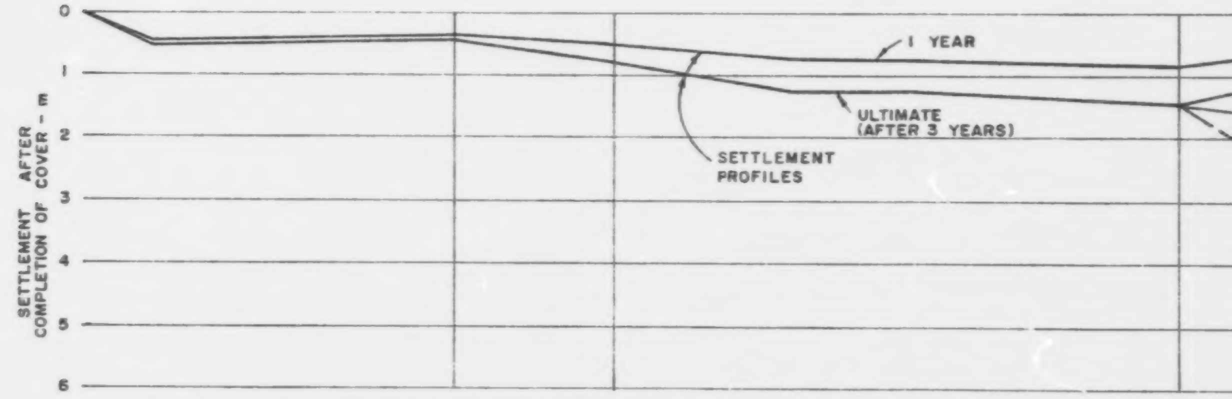
Based on the geometry and zoning of the tailings deposit in section shown in Figures 2-4 and 2-5, and the results of the one-dimensional modelling analyses, settlement profiles for the case of deferred cover construction were prepared as shown in Figures 5-9 and 5-10 on the following pages.

If no thawing of the frozen tailings in the north beach area takes place, the predicted ultimate settlements are anticipated to result in relatively minor changes in average surface gradients. The areas experiencing more significant differential settlements are the transitions between the clarification pond and the beach areas, and the fringes of the tailings deposit, particularly the areas adjacent to the South Dam and along the natural ridge on the east side, where the thickness of the tailings decreases rapidly. The approximate maximum change in surface gradient adjacent to the South Dam is estimated to be less than 0.3 percent. The changes elsewhere are considerably less than this. These changes in surface gradient, although not negligible, are small compared to the natural surface slopes of the tailings surface existing prior to decommissioning. They also tend to only increase the pre-existing surface gradients rather than reverse them. Therefore, no detrimental effects are anticipated from these predicted long term settlements. Design of surface drainage and erosion protection measures can include provisional allowances to accommodate such possible changes in surface gradients.

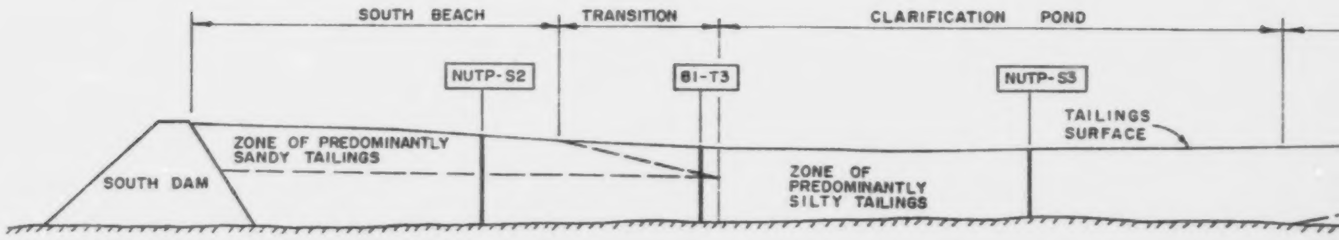
On the other hand, if complete thawing of the frozen tailings did take place, reversal of surface gradients in the transition between the north beach area and the clarification pond area could take place. In addition, settlements will tend to be highly irregular and unpredictable due to local variations in the amount of entrained ice and frozen tailings. It is therefore not possible to accommodate the effects of such an event in design. For the present, it is sufficient to note the possible consequences of such an eventuality.



NO DESICCATION - DEFERRED COVER CONSTRUCTION



FULL DESICCATION - DEFERRED COVER CONSTRUCTION


SOUTH BEACH
TRANSITION
CLARIFICATION POND
NUTP-S2
81-T3
NUTP-S3
TAILINGS SURFACE
ZONE OF PREDOMINANTLY SANDY TAILINGS
SOUTH DAM
ZONE OF PREDOMINANTLY SILTY TAILINGS

TWO - DIMENSIONAL MODEL (SECTION A - A FIGURE 2-3)

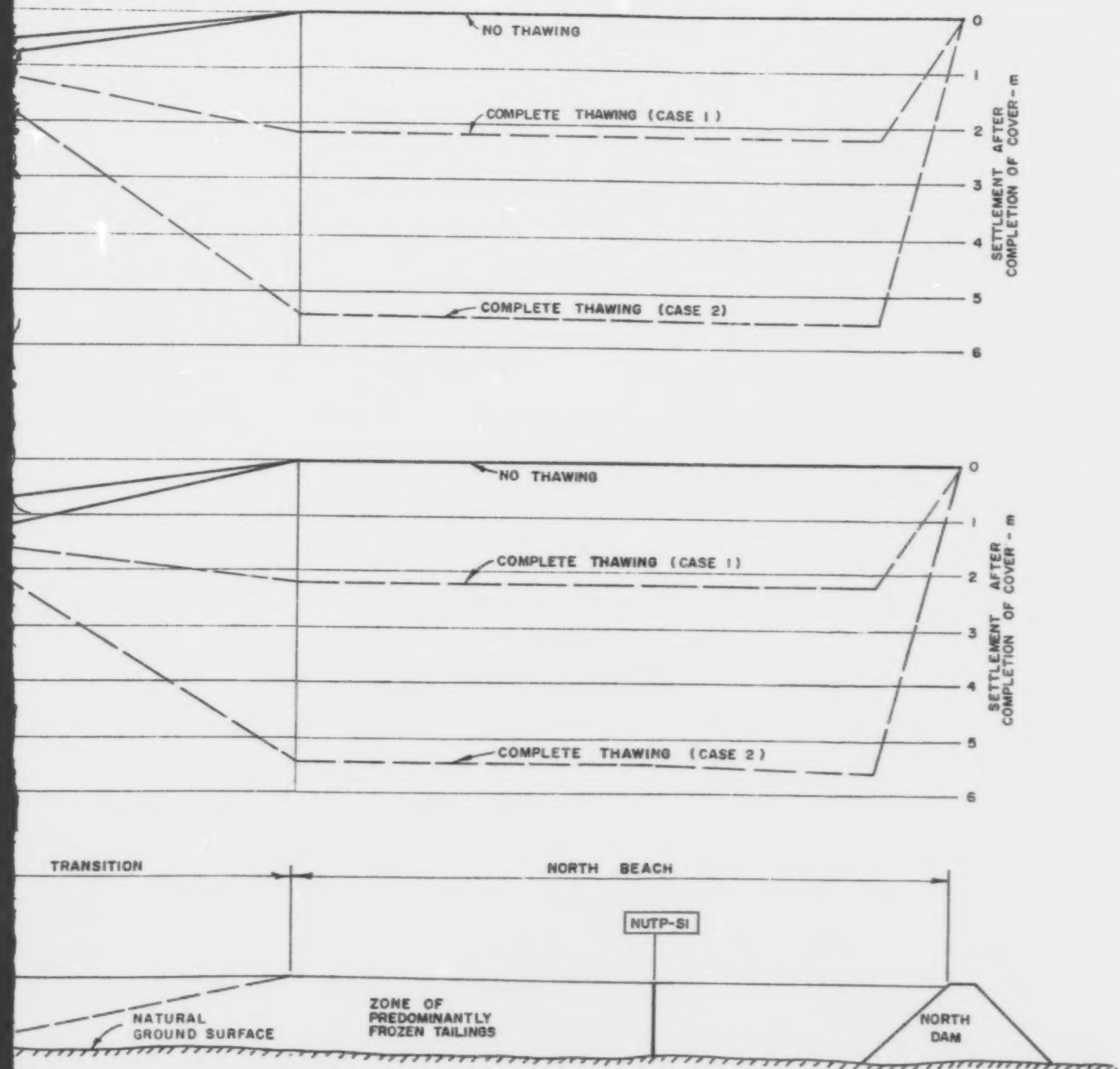
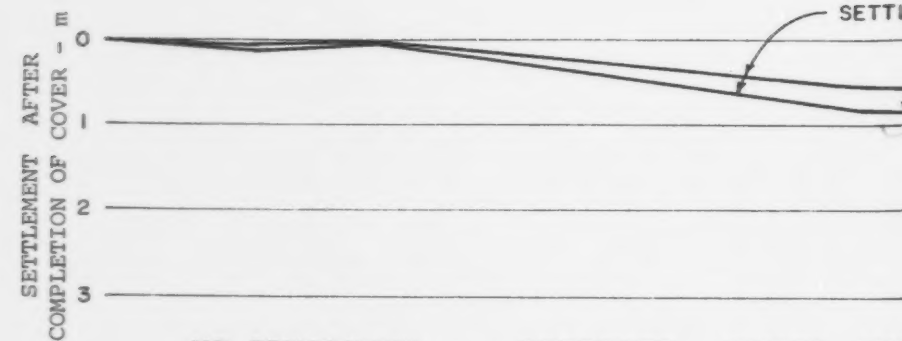
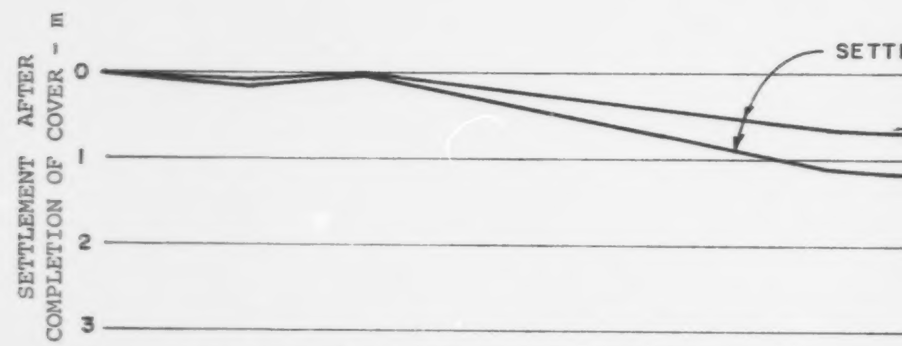


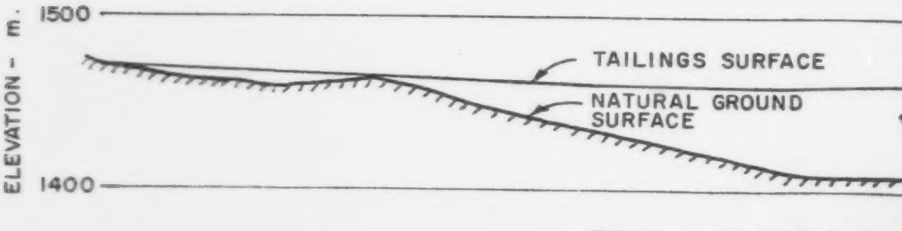
FIGURE 5 - 9
RABBIT LAKE PROJECT
SETTLEMENT PROFILES



NO DESICCATION - DEFERRED COVER CO



FULL DESICCATION - DEFERRED COVER C


ELEVATION - m.
1500
1400
TAILINGS SURFACE
NATURAL GROUND SURFACE

TWO - DIMENSION

(SECTION B - B FIGURE

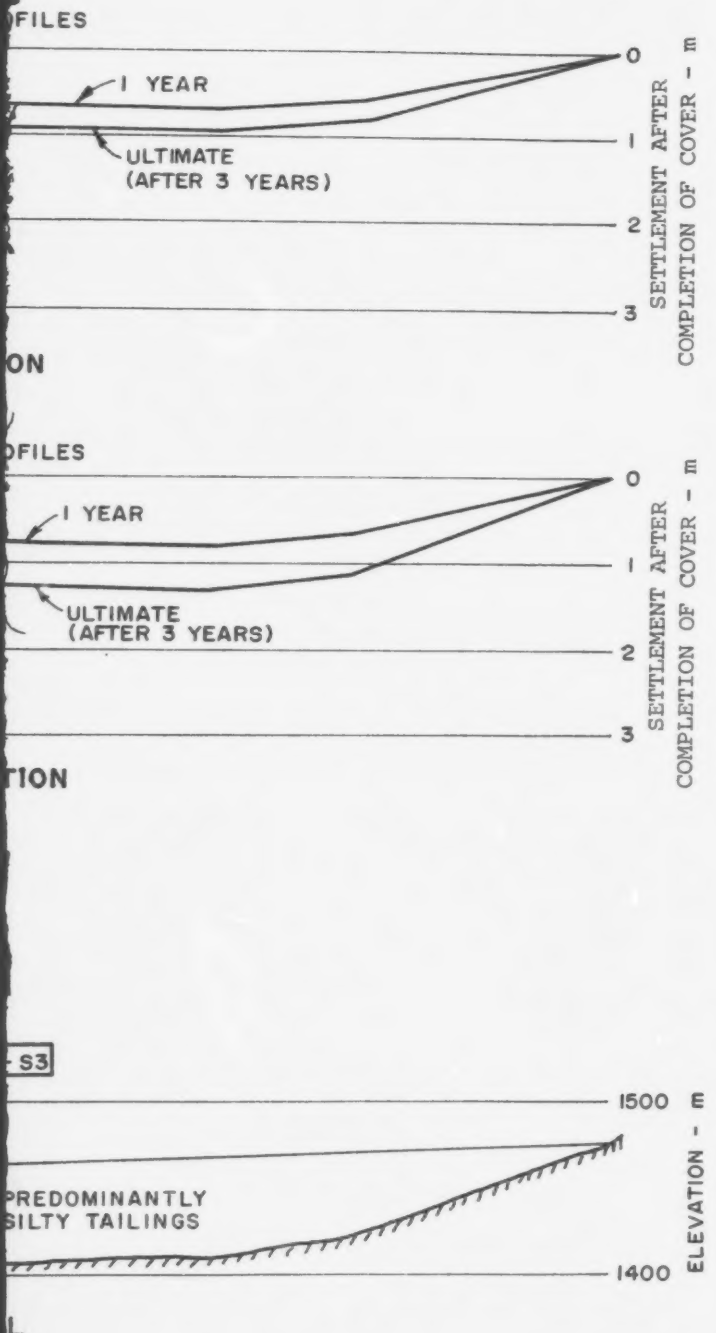


FIGURE 5 - 10
RABBIT LAKE PROJECT
SETTLEMENT PROFILES

6.0 CONCLUSION

A site specific investigation on the geotechnical properties of the Rabbit Lake tailings has been successfully completed in accord with the terms of reference for this part of the study. The results of Part A of the study provided guidance to focus on the essential parameters for the investigation, placing particular emphasis on the importance of the in-situ void ratio profile as the most appropriate characteristic representing the overall average compressibility of a stratified tailings deposit.

The results of the investigation, supplemented by selected data from pre-existing information reported by Gulf Minerals (1981b), enabled idealized one-dimentional modelling of the unfrozen part of the tailings deposit to be made to predict its long term consolidation/desiccation behaviour in relation to the planning of decommissioning strategies.

The investigation also identified the northern part of the tailings deposit as being predominantly frozen. It is believed that the frozen tailings will remain frozen as permafrost except in the event of a global climatic change.

Under prevailing climatic conditions, the results of the modelling study indicate that a deferred cover construction strategy could be considered for control of long term cover settlements by encouraging or preinducing consolidation of the tailings by desiccation effects. The natural surface gradients of the tailings already existing are favourable to promote surface drainage for effective utilization of desiccation by evaporation. This method therefore involves very little cost directly related to promoting the consolidation of the tailings deposit and has clear advantage over other methods such as well or wellpoint dewatering methods discussed in Part A of the study. Provisional measures for further improving surface drainage and the effect of evaporation described in this report can be carried out using mainly manpower and equipment available on site from the operating mine at nominal cost.

Should a warming climate change cause the frozen tailings in the northern part of the deposit to thaw, large surface settlements could ultimately occur causing possible reversal in surface gradients and disruption of surface drainage facilities. The probability of this occurrence in the foreseeable future is low and the rate of surface settlement in that event will be very slow. It is therefore more appropriate to make allowance for carrying out maintenance or remedial measures if and when such an event developed rather than attempting to accommodate such large settlements in the design of decommissioning measures.

The scope of the present study is limited to characterisation of the major zones of the tailings deposit only. Definition in detail of variations within each zone by carrying out an additional drilling program will evidently be required in designing the decommissioning measures. Further work in gaining better understanding of the desiccation behaviour of the tailings through field scale experiments or observations under site specific climatic conditions, during the early stages of decommissioning, would also be very helpful in refining or improving the accuracy of predictive modelling. This includes the following major items:

- installation of a class A evaporation pan at the site
- correlation of evaporation rate from tailings surface to class A pan evaporation rates
- field estimation of saturation limit (e_{DL}) and desiccation limit (e_{SL}) of tailings
- field estimation of maximum depth of desiccation, and
- monitoring of surface settlement and porewater pressure changes after draining the clarification pond.

Test work carried out using the Nilcon vane and piezometer equipment generally confirmed the anticipated low shear strengths of the hydraulically deposited tailing and a general trend of increasing strength with depth as a result of self-weight consolidation. The near fluid condition of the surface of the tailings in the clarification pond area at the end of deposition is a principal reason for the need to defer cover placement for surface desiccation to take place.

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APPENDIX I

BOREHOLE LOGS AND LABORATORY TEST RESULTS

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BORING LOG

PROJECT DSS - URANIUM TAILINGS CONSOLIDATION STUDY
BORING NUTP-S1
PAGE 1 OF 3
CONTRACT NO. 43139
BORING DATE Feb 5-6/86
DATUM GEODETIC
CASING NW

SAMPLE CONDITION
☒ DISTURBED
▨ GOOD
■ LOST

SAMPLE TYPES
SS SPLIT SPOON
ST THIN WALLED OPEN (SHELBY)
PS PISTON SAMPLER
WS WASH SAMPLE
RC ROCK CORE

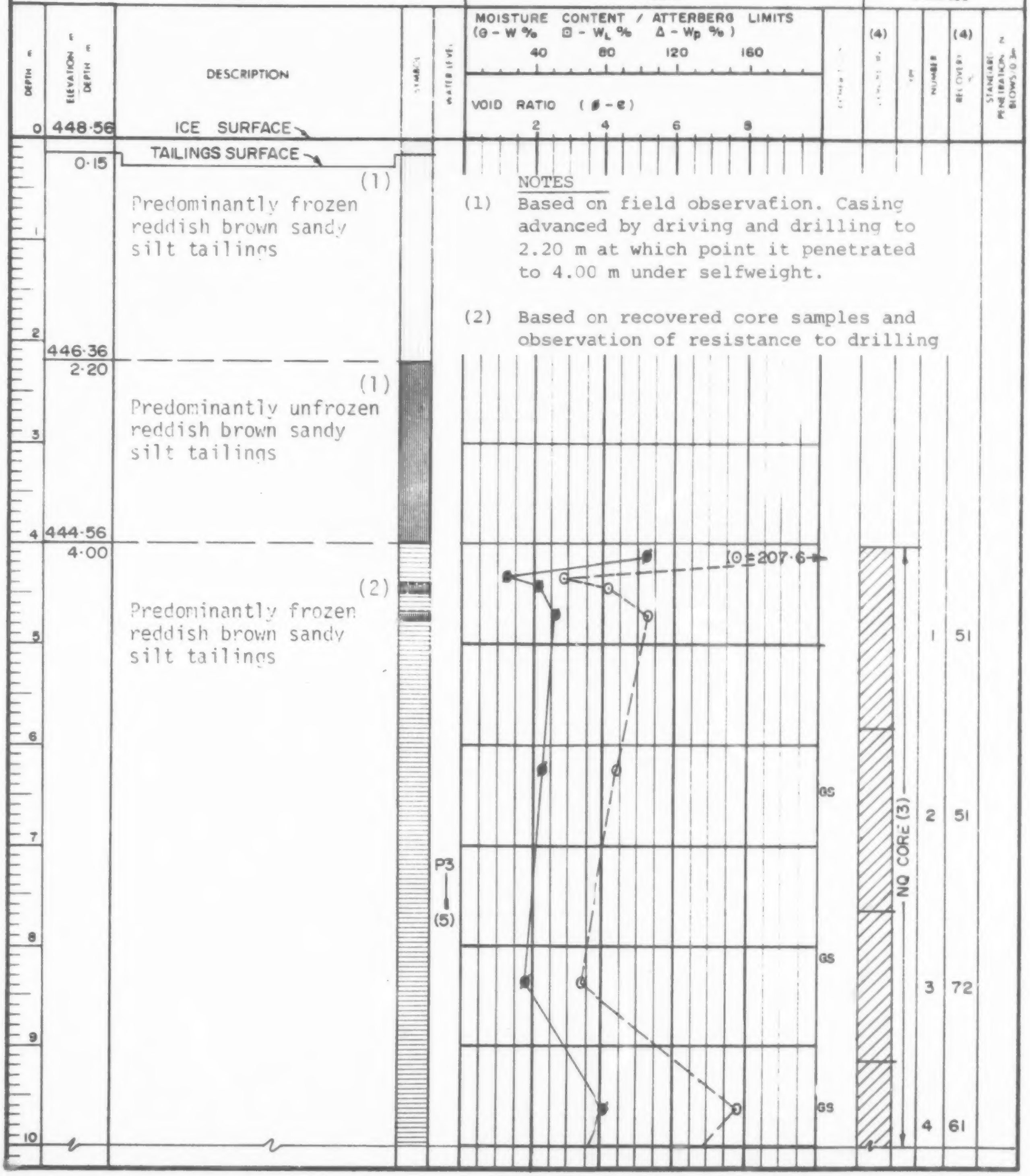
ABBREVIATIONS
GS - GRAIN SIZE ANALYSIS
γ_D - DRY UNIT WEIGHT kN/m^3
C - CONSOLIDATION
SG - SPECIFIC GRAVITY
k - PERMEABILITY cm/sec.
DS - DIRECT SHEAR
Q - TRIAXIAL, QUICK

Elevation / Depth (m)	Description
448·56 / 0	ICE SURFACE
0·15	TAILINGS SURFACE
0·15	(1) Predominantly frozen reddish brown sandy silt tailings
446·36 / 2·20	(1) Predominantly unfrozen reddish brown sandy silt tailings
444·56 / 4·00	(2) Predominantly frozen reddish brown sandy silt tailings

NOTES

(1) Based on field observation. Casing advanced by driving and drilling to 2.20 m at which point it penetrated to 4.00 m under selfweight.

(2) Based on recovered core samples and observation of resistance to drilling



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BORING LOG

PROJECT DSS - URANIUM TAILINGS CONSOLIDATION STUDY
BORING NUTP-SI
PAGE 2 OF 3
CONTRACT NO. 43139
BORING DATE Feb. 5-6/86
DATUM GEODETIC
CASING NW

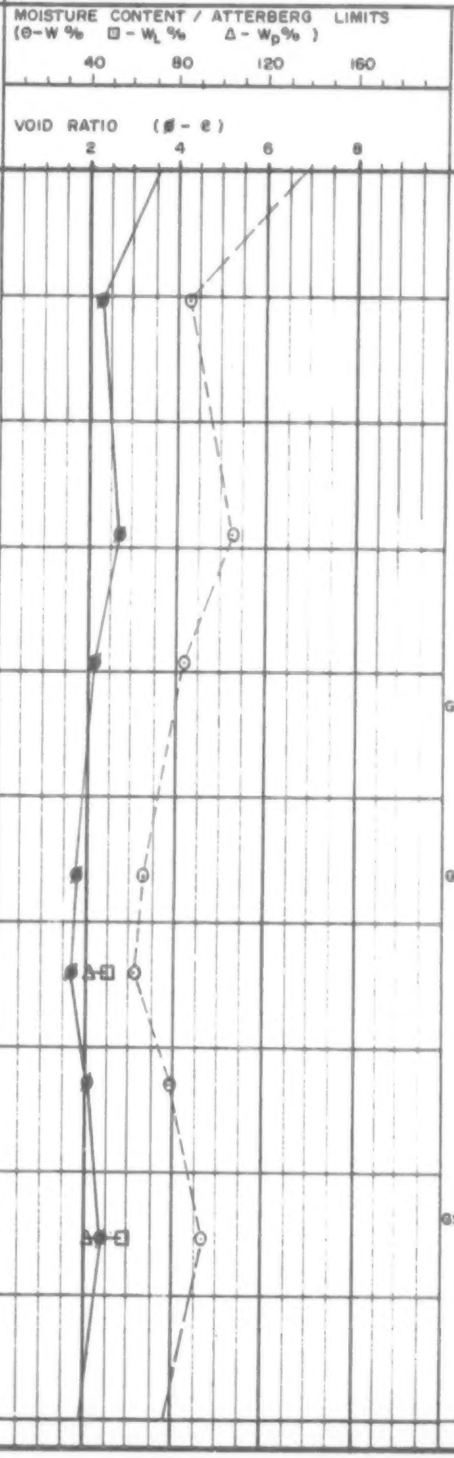
SAMPLE CONDITION
☒ DISTURBED
▨ GOOD
■ LOST

SAMPLE TYPES
SS SPLIT SPOON
ST THIN WALLED OPEN (SHELBY)
PS PISTON SAMPLER
WS WASH SAMPLE
RC ROCK CORE

ABBREVIATIONS
GS - GRAIN SIZE ANALYSIS
γ_D - DRY UNIT WEIGHT kN/m^3
C - CONSOLIDATION
SG - SPECIFIC GRAVITY
k - PERMEABILITY cm/sec.
DS - DIRECT SHEAR
Q - TRIAXIAL, QUICK

STRATIGRAPHY / TESTS / SAMPLES

DEPTH m	ELEVATION m / DEPTH m	DESCRIPTION	SYMBOL	WATER LEVEL	OTHER TESTS	CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION N BLOWS 0.3m
10	438·56 10·00	CONTINUED				▨	NQ CORE (3)	4	61	
11		(2) Predominantly frozen reddish brown sandy silt tailings				▨		5	93	
12				P2 (5)						
13						▨		6	93	
14					GS	▨		7	71	
15										
16					GS	▨		8	89	
17	431·81 16·75	(2) Predominantly unfrozen reddish brown sandy silt tailings				▨		9	5	
18					GS	▨				
19						▨		10	68	
20										



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BORING LOG

PROJECT DSS - URANIUM TAILINGS CONSOLIDATION STUDY

BORING NUTP-S1

PAGE 3 OF 3

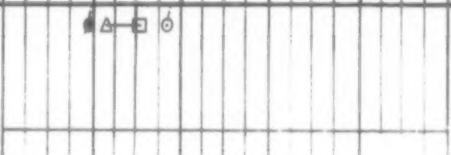
CONTRACT NO. 43139

BORING DATE Feb. 5-6/86

DATUM GEODETIC

CASING NW

SAMPLE CONDITION	SAMPLE TYPES	ABBREVIATIONS
☒ DISTURBED ▨ GOOD ■ LOST	SS SPLIT SPOON ST THIN WALLED OPEN (SHELBY) PS PISTON SAMPLER WS WASH SAMPLE RC ROCK CORE	GS - GRAIN SIZE ANALYSIS γ_D - DRY UNIT WEIGHT kN/m^3 C - CONSOLIDATION SG - SPECIFIC GRAVITY k - PERMEABILITY cm/sec. DS - DIRECT SHEAR Q - TRIAXIAL, QUICK

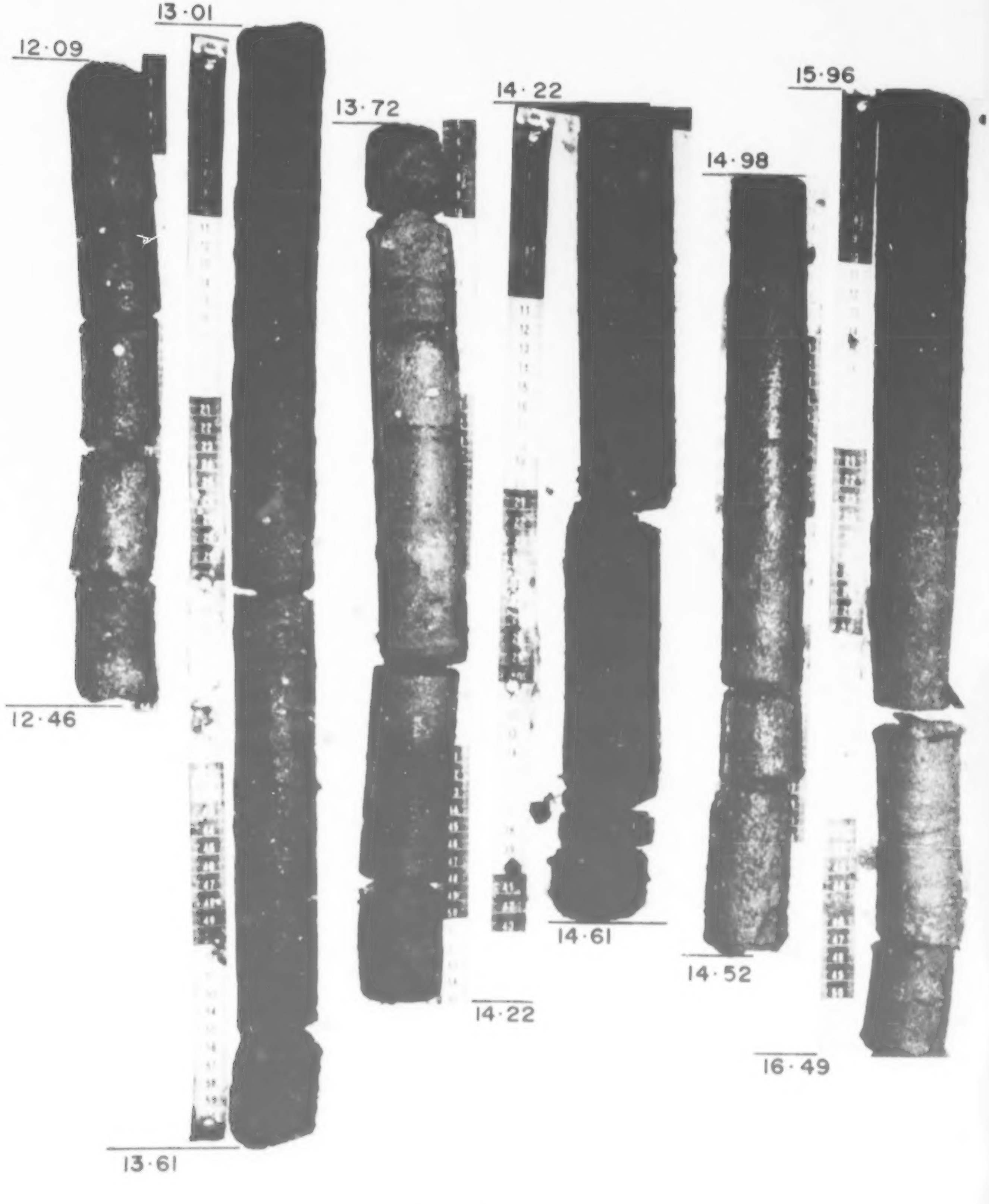
STRATIGRAPHY					TESTS		SAMPLES				
DEPTH m	ELEVATION m DEPTH m	DESCRIPTION	SYMBOL	WATER LEVEL	MOISTURE CONTENT / ATTERBERG LIMITS (○ - W % ⊟ - W_L % △ - W_p %) 40 80 120 160 VOID RATIO (● - e) 2 4 6 8	OTHER TESTS	CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION BLOWS/0.3m
20	428·56	CONTINUED									
	20·00 427·38	(2) Predominantly unfrozen tailings	▤	PI		GS	▨	NQ CORE	11	36	
21	21·18	END OF BOREHOLE		(5)							
22											

NOTES (continued)

(3) Core sampling carried out using NQ size triple tube core barrel with chilled drilling fluid of brine solution.

(4) Actual depth intervals where core loss occurred were not recorded due to practical difficulties in detecting random thin layers of unfrozen tailings interbedded with more predominant layers of frozen tailings.

(5) For details of piezometers and water level readings see report text.


12·09
12·46
13·01
13·61
13·72
14·22
14·22
14·61
14·98
14·52
15·96
16·49

BOREHOLE S1

Geocon

BORING LOG

PROJECT DSS-URANIUM TAILINGS CONSOLIDATION STUDY
BORING NUTP-S2
PAGE 1 OF 3
CONTRACT NO. 43139
BORING DATE Feb. 14-16/86
DATUM GEODETIC
CASING HW

SAMPLE CONDITION	SAMPLE TYPES
☒ DISTURBED	SS SPLIT SPOON
▨ GOOD	ST THIN WALLED OPEN (SHELBY)
■ LOST	PS PISTON SAMPLER 76 mm
	WS WASH SAMPLE
	RC ROCK CORE

ABBREVIATIONS
GS - GRAIN SIZE ANALYSIS
γ_D - DRY UNIT WEIGHT kN/m^3
C - CONSOLIDATION
SG - SPECIFIC GRAVITY
k - PERMEABILITY cm/sec.
DS - DIRECT SHEAR
Q - TRIAXIAL, QUICK

STRATIGRAPHY

DEPTH m	ELEVATION m / DEPTH m	DESCRIPTION
0	448.48	ICE SURFACE
	0.00	Ice
1	447.48 1.00	Water
2	446.35	Tailings surface(1)
	2.13	Frozen tailings
	2.33	Reddish brown sandy silt to silt tailings

TESTS

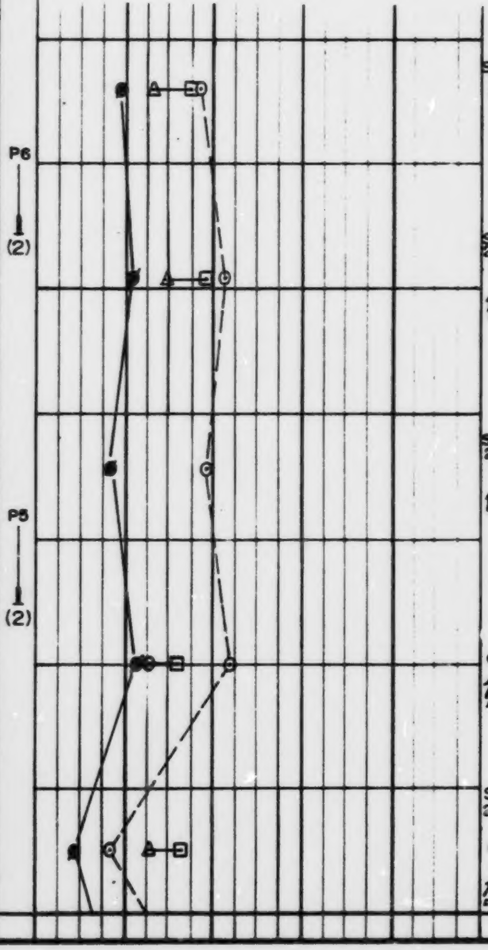
MOISTURE CONTENT / ATTERBERG LIMITS (⊙ - W % ⊟ - W_L% Δ - W_p%) 40 80 120 160

VOID RATIO (ø - e) 2 4 6 8

NOTES

(1) Tailings surface determined by sounding and confirmed by field hydrometer and slurry density tests. Refer to report text for details.

(2) For details of piezometers and water level readings, see report text.



SAMPLES

[illegible]	CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION N BLOWS/0.3m
SG 2.54 GS	▨	PS	1	100	HYDRAULIC PUSH
SG 2.57 GS γ_D=8.6	▨	PS	2	100	
SG 2.57 GS C γ_D=8.94	▨	PS	3	100	
GS γ_D=7.85	▨	PS	4	100	
SG 2.59 GS C γ_D=12.22	▨	PS	5	100	

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BORING LOG

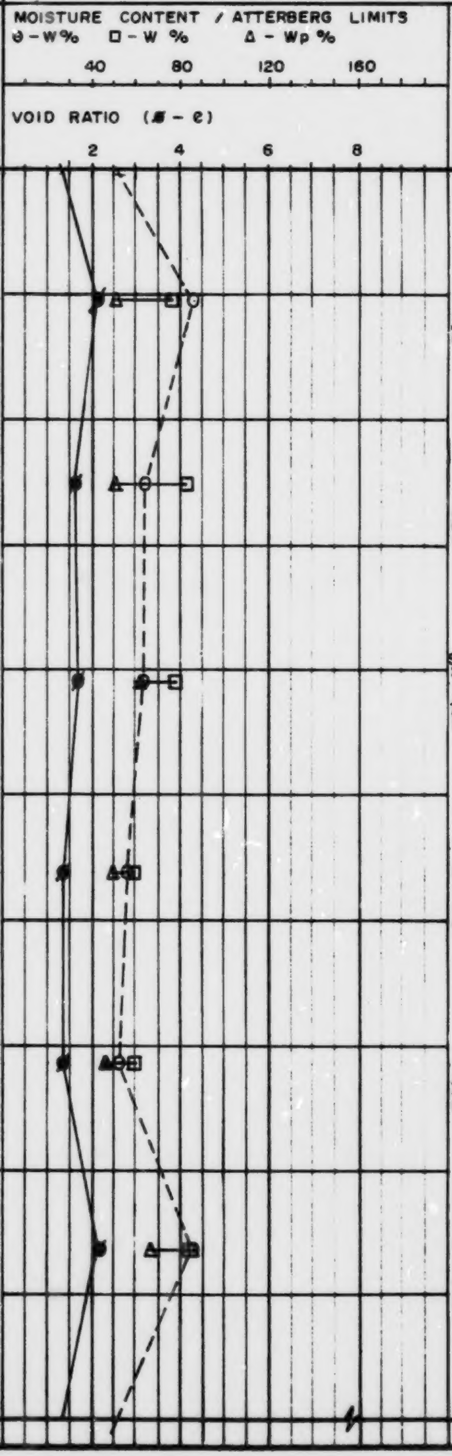
PROJECT DSS - URANIUM TAILINGS CONSOLIDATION STUDY
BORING NUTP S2
PAGE 2 OF 3
CONTRACT NO. 43139
BORING DATE Feb.14-16/86
DATUM GEODETIC
CASING HW

SAMPLE CONDITION	SAMPLE TYPES	ABBREVIATIONS
☒ DISTURBED ▨ GOOD ■ LOST	SS SPLIT SPOON ST THIN WALLED OPEN (SHELBY) PS PISTON SAMPLER 76 mm WS WASH SAMPLE RC ROCK CORE	GS - GRAIN SIZE ANALYSIS γ_D - DRY UNIT WEIGHT kN/m^3 C - CONSOLIDATION SG - SPECIFIC GRAVITY k - PERMEABILITY cm/sec. DS - DIRECT SHEAR Q - TRIAXIAL, QUICK

STRATIGRAPHY

DEPTH m	ELEVATION m / DEPTH m	DESCRIPTION	SYMBOL	WATER LEVEL
10	438·48	CONTINUED		
	10·00	Reddish brown sandy silt to silt tailings	■	
11				
12				P4 (2)
13				
14				
15				
16				P3 (2)
17				
18				
19				P2 (2)
20				

TESTS



SAMPLES

OTHER TESTS	CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION N BLOWS/0.3m
GS γ_D=7·67	▨	PS	6	100	HYDRAULIC PUSH
C γ_D=10·19	▨	PS	7	100	
SG 2·61 GS γ_D=10·73	▨	PS	8	100	
GS C γ_D=10·38	▨	PS	9	100	
γ_D=11·16	▨	PS	10	100	
GS C γ_D=7·8[illegible]	▨	PS	11	100	
	▨				

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BORING LOG

PROJECT DSS - URANIUM TAILINGS CONSOLIDATION STUDY

BORING NUTP-S2

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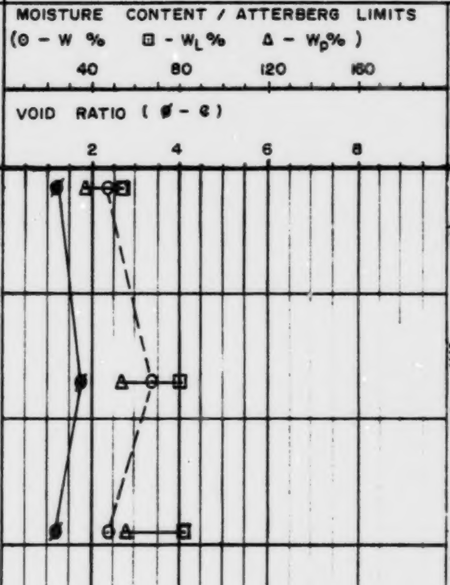
DATUM GEODETIC

CASING HW

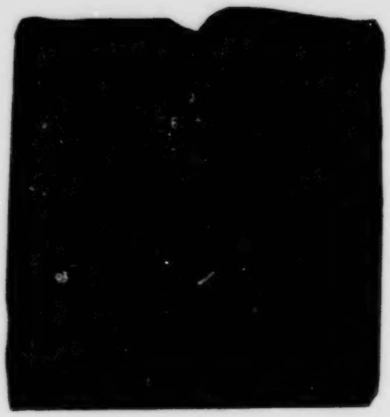
SAMPLE CONDITION
☒ DISTURBED
▨ GOOD
■ LOST

SAMPLE TYPES
SS SPLIT SPOON
ST THIN WALLED OPEN (SHELBY)
PS PISTON SAMPLER 76 mm
WS WASH SAMPLE
RC ROCK CORE

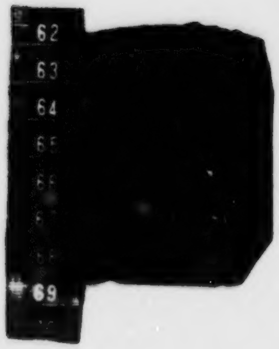
ABBREVIATIONS
GS - GRAIN SIZE ANALYSIS
γ_D - DRY UNIT WEIGHT kN/m^3
C - CONSOLIDATION
SG - SPECIFIC GRAVITY
k - PERMEABILITY cm/sec.
DS - DIRECT SHEAR
Q - TRIAXIAL, QUICK-



STRATIGRAPHY					TESTS	SAMPLES				
DEPTH m	ELEVATION m / DEPTH m	DESCRIPTION	SYMBOL	WATER LEVEL	OTHER TESTS	CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION N BLOWS 0.3m
20	428·48	CONTINUED								
	20·00	Reddish brown sandy silt to silt tailings	■		GS γ_D 9·99	▨	PS	12	100	HYDRAULIC PUSH
21										
22				PI	SG 2·57 GS C γ_D 9·00	▨	PS	13	100	
23	425·19			(2)	GS γ_D 9·94	▨	PS	14	100	
	23·29	END OF BOREHOLE								
24										
25										



SAMPLE A (7.62m - 8.38m)



SAMPLE 5 (9.14m - 9.91m)

62
63
64
65
69

SAMPLE 6 (10.67m - 11.43m)

62
63
64
65
66
67
68
69
70

SAMPLE 10 (16.76m - 17.53m)

BOREHOLE S2

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BORING LOG

PROJECT DSS - URANIUM TAILINGS
CONSOLIDATION STUDY
BORING NUTP-S3
PAGE 1 OF 3
CONTRACT NO. 43139
BORING DATE Feb. 10-12/86
DATUM GEODETIC
CASING HW

SAMPLE CONDITION
☒ DISTURBED
▨ GOOD
■ LOST

SAMPLE TYPES
SS SPLIT SPOON
ST THIN WALLED OPEN (SHELBY)
PS PISTON SAMPLER 76 m m
WS WASH SAMPLE
RC ROCK CORE

ABBREVIATIONS
GS - GRAIN SIZE ANALYSIS
γ_d - DRY UNIT WEIGHT kN/m^3
C - CONSOLIDATION
SG - SPECIFIC GRAVITY
k - PERMEABILITY (x 10^{-4} cm/sec.)
DS - DIRECT SHEAR
Q - TRIAXIAL, QUICK

STRATIGRAPHY

DEPTH m	ELEVATION m / DEPTH m	DESCRIPTION	SYMBOL	WATER LEVEL
0	448·55	ICE SURFACE		
	0·00	ICE		
1	447·55 1·00	WATER		
4	444·59 3·96	TAILINGS SURFACE (1)		P5 (2)
		Reddish brown sandy silt to clayey silt tailings	■	P4 (2)

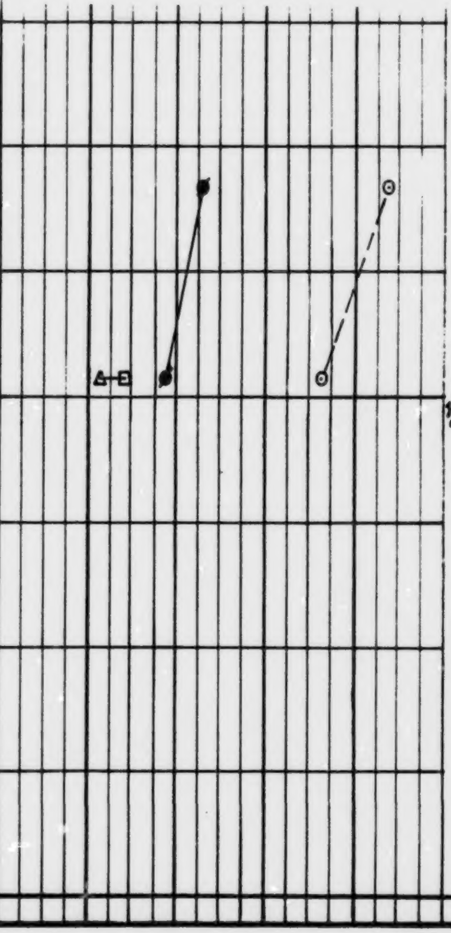
NOTES

(1) Tailings surface determined by sounding and confirmed by field hydrometer and slurry density tests. Refer to report text for details.

(2) For details of piezometers and water level readings, see report text.

TESTS

MOISTURE CONTENT ATTERBURG LIMITS
(○ - W % □ - W_L % Δ - W_P %)
40 80 120 160
VOID RATIO (● - e)
2 4 6 8



OTHER TESTS: GS
γ_d 5·82
e

SAMPLES

CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION N BLOWS/0.3m
☒	PS	1	50	HYDRAULIC PUSH
☒	PS	2	33	HYDRAULIC PUSH
☒	PS	3	60	HYDRAULIC PUSH
■	PS	4	0	HYDRAULIC PUSH
■	PS	5	0	HYDRAULIC PUSH
■	PS	6	0	HYDRAULIC PUSH
■	PS	7	0	HYDRAULIC PUSH
■	PS	8	0	HYDRAULIC PUSH

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BORING LOG

PROJECT D S S - URANIUM TAILINGS CONSOLIDATION STUDY

BORING NUTP-S3

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CONTRACT NO. 43139

BORING DATE Feb. 10-12/86

DATUM GEODETIC

CASING HW

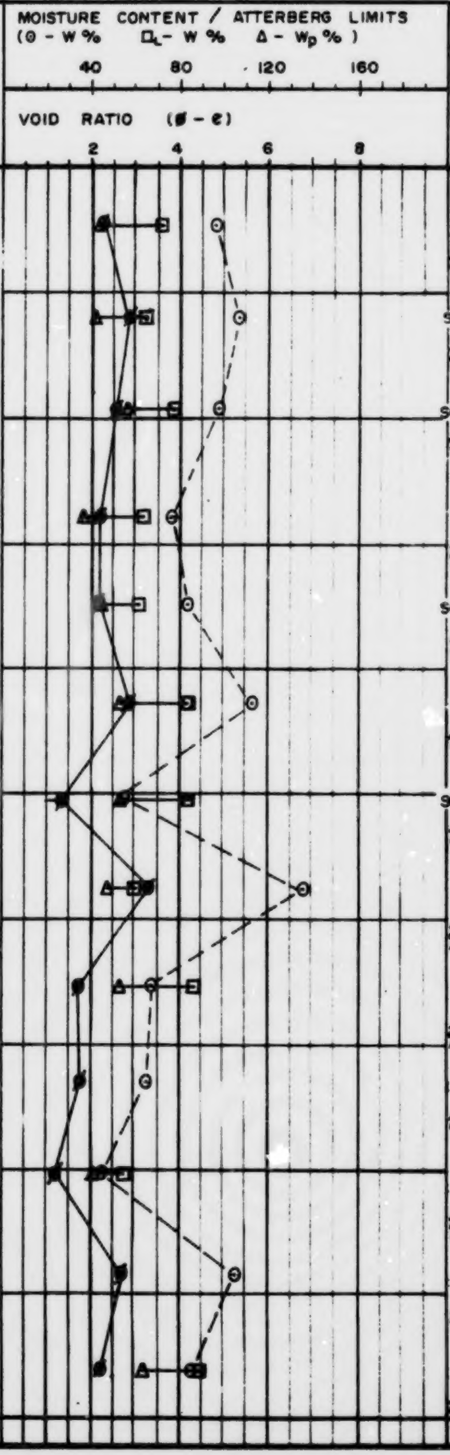
SAMPLE CONDITION	SAMPLE TYPES	ABBREVIATIONS
☒ DISTURBED ▨ GOOD ■ LOST	SS SPLIT SPOON ST THIN WALLED OPEN (SHELBY) PS PISTON SAMPLER 76 mm WS WASH SAMPLE RC ROCK CORE	GS - GRAIN SIZE ANALYSIS γ_D - DRY UNIT WEIGHT kN/m^3 C - CONSOLIDATION SG - SPECIFIC GRAVITY k - PERMEABILITY (x 10^{-4} cm/sec) DS - DIRECT SHEAR Q - TRIAXIAL, QUICK

STRATIGRAPHY

DEPTH m	ELEVATION m / DEPTH m	DESCRIPTION
10	438.55 10.00	CONTINUED
10–20		Reddish brown sandy silt to clayey silt tailings

WATER LEVEL: P3 (2); P2 (2); P1

TESTS



SAMPLES

OTHER TESTS	CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION N BLOWS/0.3m
GS γ_D=7.32	▨	PS	9	100	HYDRAULIC PUSH
C SG 2.63 GS γ_D=7.45	▨	PS	10	100	HYDRAULIC PUSH
SG 2.62 GS γ_D=7.21	▨	PS	11	100	HYDRAULIC PUSH
GS γ_D=9.34	▨	PS	12	100	HYDRAULIC PUSH
C SG 2.55 GS γ_D=7.84	▨	PS	13	100	HYDRAULIC PUSH
GS γ_D=8.11	▨	PS	14	100	HYDRAULIC PUSH
SG 2.68 GS γ_D=10.81	▨	PS	15	100	HYDRAULIC PUSH
C GS γ_D=6.79	▨	PS	16	100	HYDRAULIC PUSH
GS γ_D=9.36	▨	PS	17	100	HYDRAULIC PUSH
C GS γ_D=10.25	▨	PS	18	100	HYDRAULIC PUSH
GS γ_D=11.72	▨	PS	19	100	HYDRAULIC PUSH
C GS γ_D=5.91	▨	PS	20	100	HYDRAULIC PUSH
GS γ_D=7.38	▨	PS	21	100	HYDRAULIC PUSH

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BORING LOG

PROJECT DSS - URANIUM TAILINGS BORING NUTP S3

CONSOLIDATION STUDY PAGE 3 of 3

CONTRACT NO. 43139 BORING DATE Feb.10-12/86

DATUM GEODETIC CASING HW

SAMPLE CONDITION

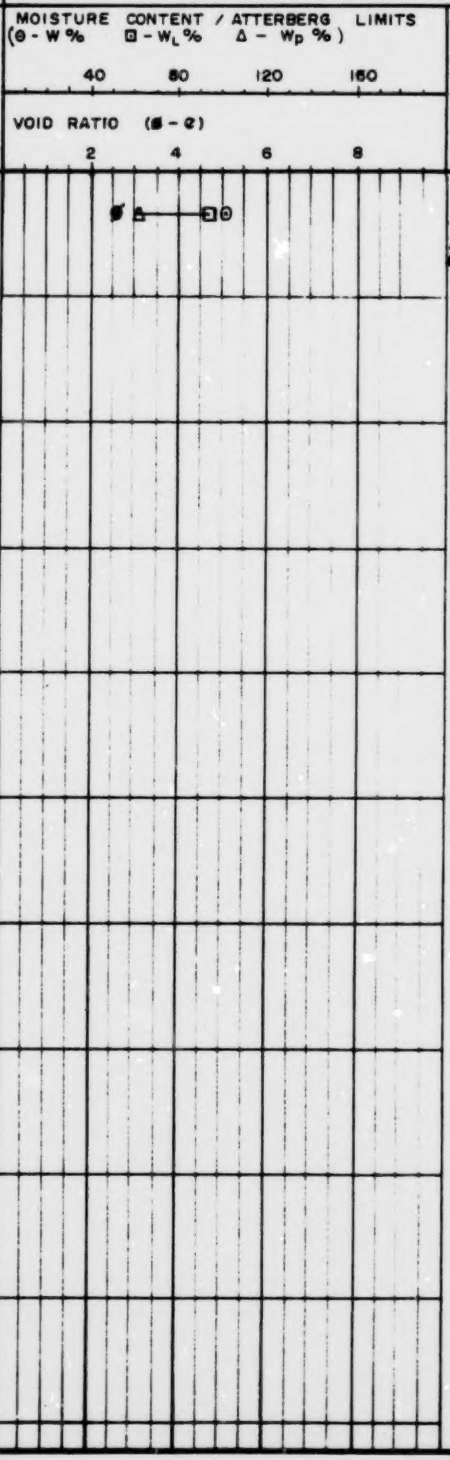
☒ DISTURBED
▨ GOOD
■ LOST

SAMPLE TYPES

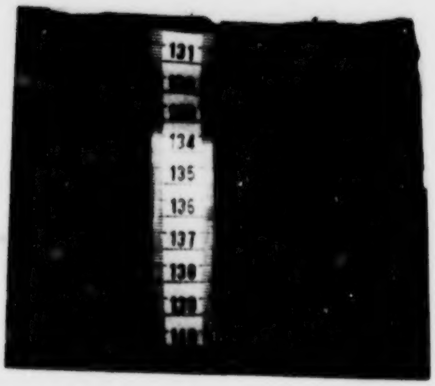
SS SPLIT SPOON
ST THIN WALLED OPEN (SHELBY)
PS PISTON SAMPLER 76 mm
WS WASH SAMPLE
R ROCK CORE

ABBREVIATIONS

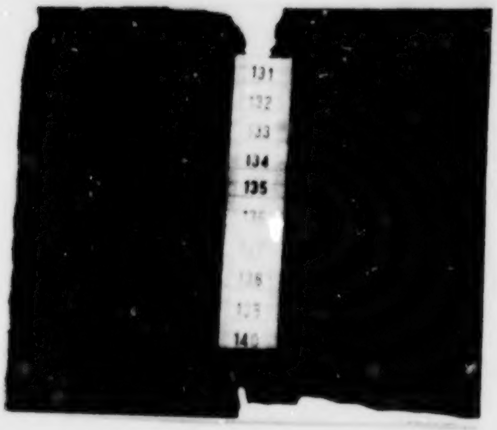
GS - GRAIN SIZE ANALYSIS
γ_D - DRY UNIT WEIGHT kN/m^3
C - CONSOLIDATION
SG - SPECIFIC GRAVITY
k - PERMEABILITY ($\times 10^{-4}$ cm/sec.)
DS - DIRECT SHEAR
Q - TRIAXIAL, QUICK



STRATIGRAPHY					TESTS	SAMPLES					
DEPTH m	ELEVATION m DEPTH m	DESCRIPTION	SYMBOL	WATER LEVEL	OTHER TESTS	CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION N BLOWS 0.3m
20	428·55	CONTINUED								
	20·00 427·83	Reddish brown tailings	■		C GS γ_D :763	▨	PS	22	100	HYD. PUSH
21	20·72	END OF BOREHOLE								
22										
23										
24										
25										
26										
27										
28										
29										
30										


131
134
135
136
137

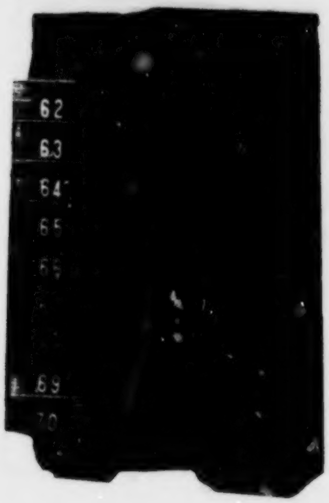
SAMPLE 4 (10.06m - 10.82m)


131
132
133
134
135
140

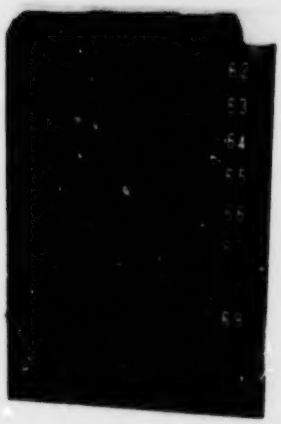
SAMPLE 5 (10,82m - 11.58m)

62
63
64
65
66

SAMPLE 11 (11.58m - 12.34m)


62
63
64
65
66
69
70

SAMPLE 17 (16.15m - 16.92m)


62
63
64
65
66
69

SAMPLE 19 (17.68m - 18.44m)

BOREHOLE S3

TABLE I-1A - SUMMARY OF LABORATORY TEST RESULTS
BOREHOLE NUTP-S1

Sample No.	Depth (m)	Moisture Content (%)	Unit Weight (KN/m^3)	Specific Gravity	Liquid Limit (%)	Plastic Limit (%)	Grain Size Distribution (% Passing)								Initial Void Ratio
							Sieve No.					Diameter (mm)			
							20	40	60	100	200	0.01	0.005	0.002	
1A	4.01- 4.33	207.6													
1B	4.33- 4.44	50.9													
1C	4.44- 4.51	81.2													
1D	4.51- 4.95	103.6			61	45									
2A	5.84- 6.74	86.5					100	100	97	91	71	26	15	6	
2B	5.84- 6.74						100	99	85	48	38	14	11	8	
3	7.62- 7.15	69.7					100	100	90	73	35	17	14	12	2.26
4	9.15-10.08	157.6					100	97	91	73	43	18	16	13	
5	10.67-11.40	86.0													
6	12,09-13.61	102.6													3.21
7	13.72-14.19	82.1					100	100	96	85	70	29	20	11	
8A	14.98-16.31	67.6													
8B	14.98-16.58	60.4			50	41	100	93	84	74	60	30	20	4	
9	16.76-16.81	79.0													
10	17.83-19.26	92.8			59	41	100	95	87	79	69	43	21	11	3.39
11	19.92-20.42	72.6			61	46	100	100	97	94	86	40	17	7	

TABLE I-1B - SUMMARY OF LABORATORY TEST RESULTS
BOREHOLE NUTP-S2

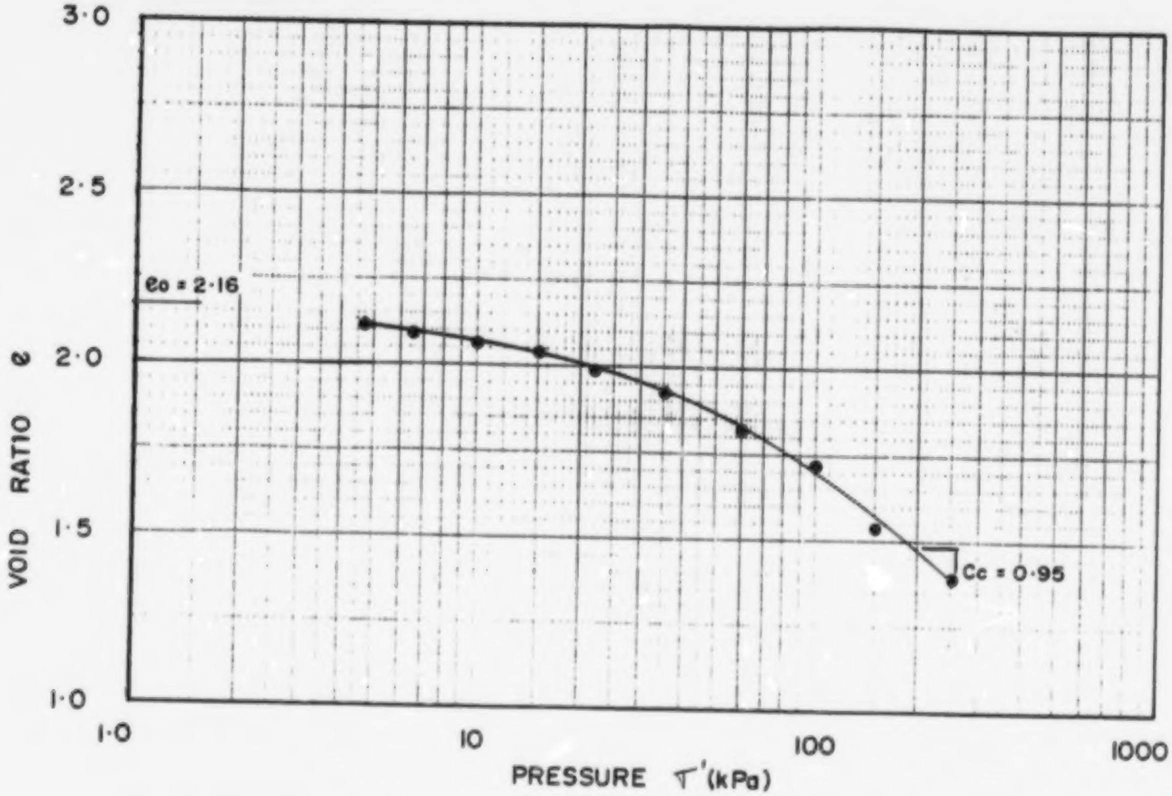
Sample No.	Depth (m)	Moisture Content (%)	Unit Weight (KN/m^3)	Specific Gravity	Liquid Limit (%)	Plastic Limit (%)	Grain Size Distribution (% Passing) Sieve No. 20	40	60	100	200	Diameter (mm) 0.01	0.005	0.002	Initial Void Ratio
1	3.05- 3.81	75.7		2.54	70	53	100	94	83	20	4	-	-	-	
							100	99	98	94	86	43	12	8	
							100	99	98	94	90	44	18	16*	
2	4.57- 5.33	86.9	16.1	2.57	67	50	100	100	100	97	92	42	29	18	4.43
							100	100	100	97	87	37	29	21*	3.71
3	6.10- 6.86	68.9	15.1	2.56			100	99	95	87	64	22	4	-	
4	7.62- 8.38	88.5	14.8		52	45	100	100	100	96	90	45	34	20	
5	9.14- 9.91	33.8	17.2		67	51	100	87	50	25	14	-	-	-	
							-	-	-	-	79	47	34	20*	
6	10.67-11.43	87.6	14.4		76	61	100	100	100	98	96	91	63	2	
7	12.19-12.95	63.0	16.6		81	51	-	-	-	-	-	-	-	-	
8	13.72-14.48	63.0	17.5		79	64	100	100	96	84	69	30	3	-	2.71
9	15.24-16.00	55.4	16.1		56	50	100	100	100	93	77	25	9	2	
10	16.76-17.53	52.8	17.0		58	46	100	100	98	88	62	25	2	-	
11	18.29-19.05	85.1	14.6		85	67	100	100	100	100	100	78	53	1	
12	19.81-20.57	47.7	14.8		54	38	100	100	96	92	83	30	3	-	
13	21.34-22.10	68.2	15.1		80	57	100	100	98	96	90	30	3	-	5.04
							-	-	-	-	92	48	37	22*	
14	22.56-23.29	47.7	14.6		81	56	100	100	97	96	95	30	3	-	

* Grain size analysis performed on specimen with most dissolved salts removed by washing.

TABLE I-1C - SUMMARY OF LABORATORY TEST RESULTS
BOREHOLE NUTP-S3

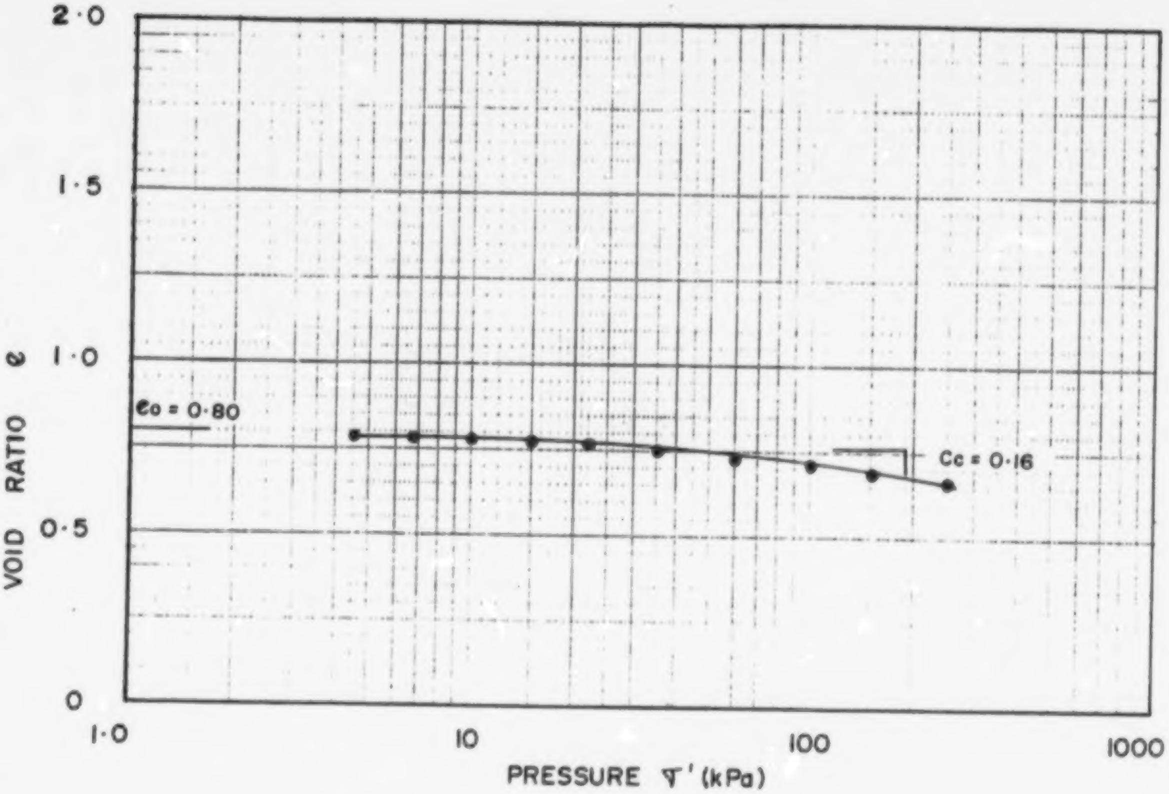
Sample No.	Depth (m)	Moisture Content (%)	Unit Weight (KN/m^3)	Specific Gravity	Liquid Limit (%)	Plastic Limit (%)	Grain Size Distribution (% Passing)								Initial Void Ratio
							Sieve No.					Diameter (mm)			
							20	40	60	100	200	0.01	0.005	0.002	
1	3.96- 4.34	175.3													
3	5.49- 5.95	145.1	14.3		58	45	100	100	100	96	89	56	44	33	
9	10.06-10.82	96.4	14.4		71	44	100	100	100	100	97	30	15	-	3.91
10	10.82-11.58	109.3	15.6	2.63	66	44	100	100	100	97	90	56	44	10	
							100	100	99	95	86	60	44	22*	
11	11.58-12.34	99.3	14.4	2.62	79	58	100	100	100	100	90	58	10	-	
							100	100	99	93	75	45	35	22*	
12	12.34-13.10	78.8	16.7		63	37	100	100	97	93	76	32	22	-	4.60
13	13.10-13.87	83.2	14.4	2.53	61	46	100	100	100	96	84	51	18	6	
14	13.87-14.63	112.9	17.1		83	53	100	100	99	96	93	40	16	4	4.00
15	14.63-15.33	55.3	16.0		83	54	100	100	100	100	90	51	8	-	
							-	-	-	-	97	56	40	19*	
16	15.33-16.15	136.0	15.6		60	49	100	100	100	96	86	22	5	-	
							-	-	-	-	94	54	30	14*	
17	16.15-16.92	69.4	15.9		87	53	100	100	100	100	98	70	44	10	
18	16.92-17.68	67.6	17.2		-	-	100	100	100	93	77	22	16	-	
19	17.68-18.44	47.5	17.3		57	43	100	97	92	86	67	27	2	-	
20	18.44-19.20	105.9	12.2		106	71	100	100	100	100	99	67	47	6	
21	19.20-19.96	87.5	13.8		87	65	100	100	100	96	93	65	46	27	
22	19.96-20.72	100.6	15.3		94	61	100	100	100	98	92	67	46	10	4.98

* Grain size analysis performed on specimen with most dissolved salts removed by washing.



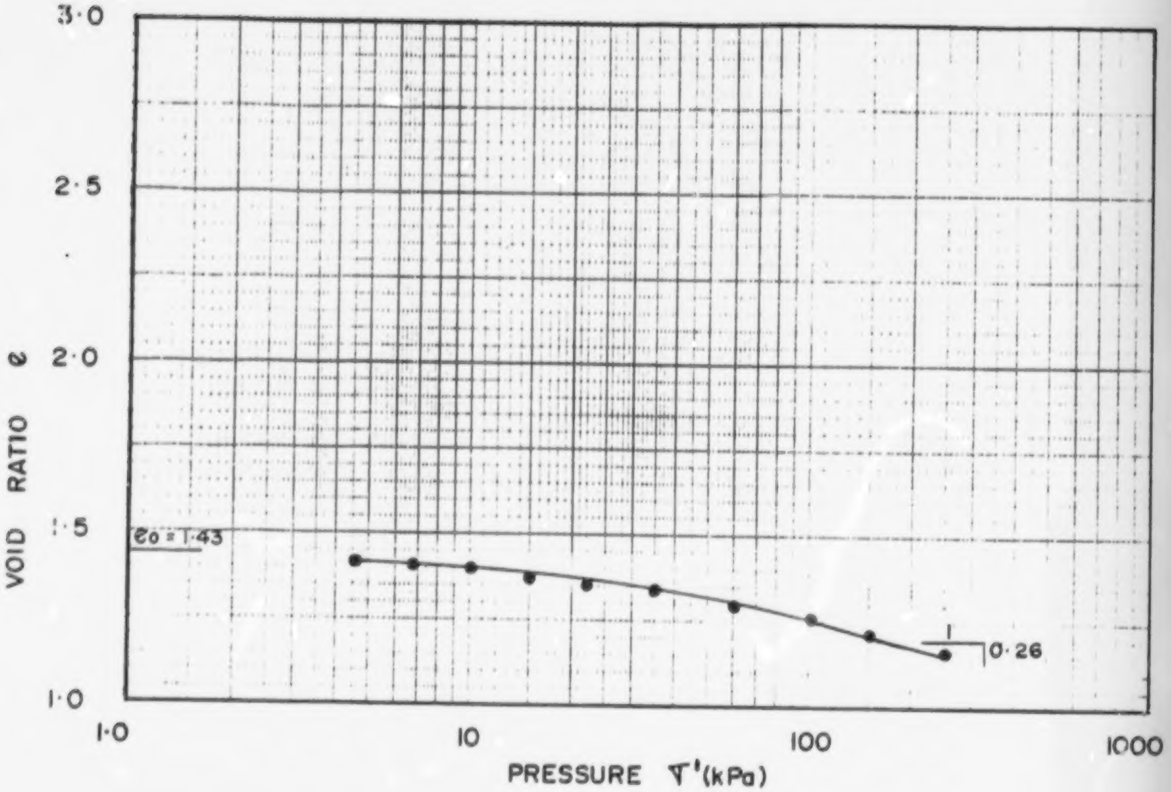
σ' (kPa)	e	C_v (m^2/day)	m_v (kPa^{-1})	k (m / sec)
0	2.16			
		0.42	2.7×10^{-3}	1.3×10^{-7}
4.5	2.12			
		0.34	4.7×10^{-3}	1.8×10^{-7}
6.75	2.09			
		0.50	3.3×10^{-3}	1.9×10^{-7}
10	2.06			
		0.99	1.4×10^{-3}	1.6×10^{-7}
15	2.04			
		0.38	2.4×10^{-3}	1.0×10^{-7}
22	1.99			
		0.11	1.7×10^{-3}	2.2×10^{-8}
35	1.92			
		0.18	1.5×10^{-3}	3.0×10^{-8}
60	1.81			
		0.54	9.3×10^{-4}	5.7×10^{-8}
100	1.71			
		0.72	1.2×10^{-3}	9.8×10^{-8}
150	1.54			
		0.33	6.1×10^{-4}	2.3×10^{-8}
250	1.39			

FIGURE I - 1 OEDOMETER TEST RESULTS
BOREHOLE NUTP-S2 Sample 3



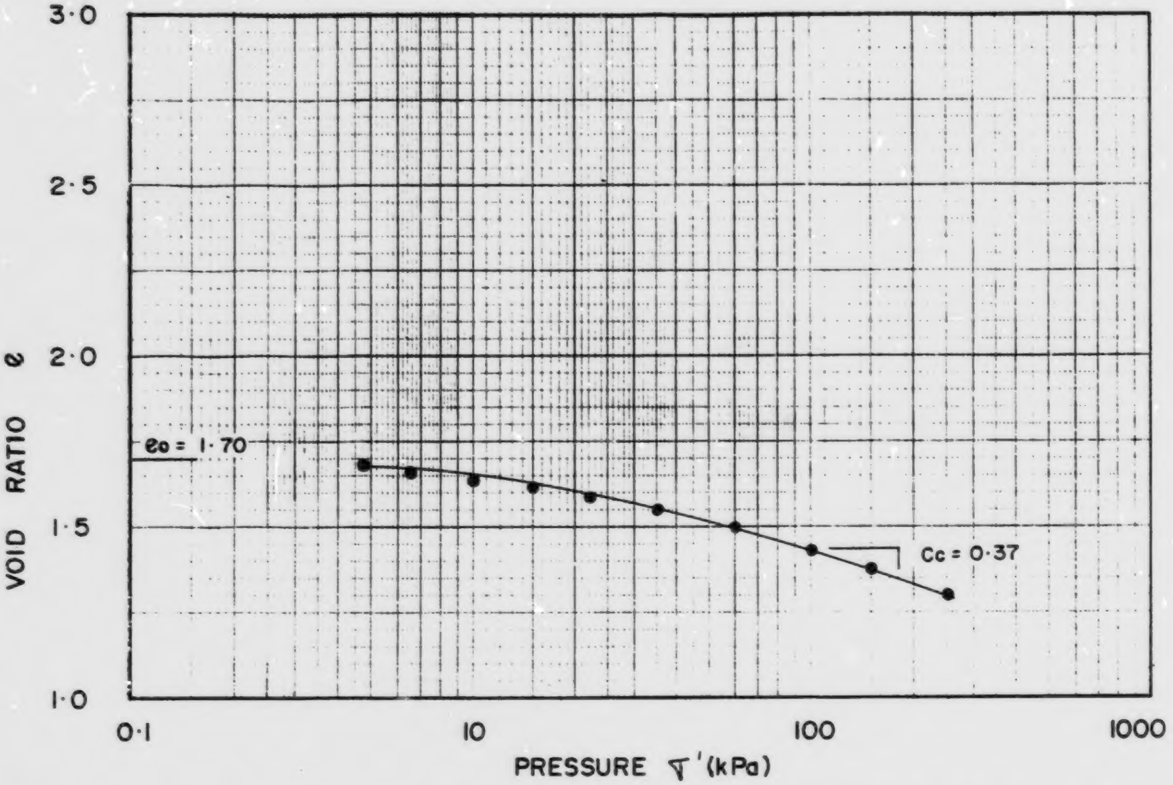
σ' (kPa)	e	c_v (m^2/day)	m_v (kPa^{-1})	k (m / sec)
0	0.80	5.29	1.1×10^{-3}	6.7×10^{-7}
4.50	0.79	1.75	2.2×10^{-3}	4.4×10^{-7}
6.75	0.78	1.36	1.8×10^{-3}	2.7×10^{-7}
10	0.78	7.40	6.8×10^{-4}	5.7×10^{-7}
15	0.77	2.55	9.7×10^{-4}	2.8×10^{-7}
22	0.76	1.12	4.4×10^{-4}	5.6×10^{-8}
35	0.75	1.11	4.1×10^{-4}	5.2×10^{-8}
60	0.73	2.43	2.6×10^{-4}	7.2×10^{-8}
100	0.71	1.60	2.0×10^{-4}	3.6×10^{-8}
150	0.69	2.32	1.7×10^{-4}	4.5×10^{-8}
250	0.66			

FIGURE I - 2 OEDOMETER TEST RESULTS
BOREHOLE NUTP-S2 Sample 5



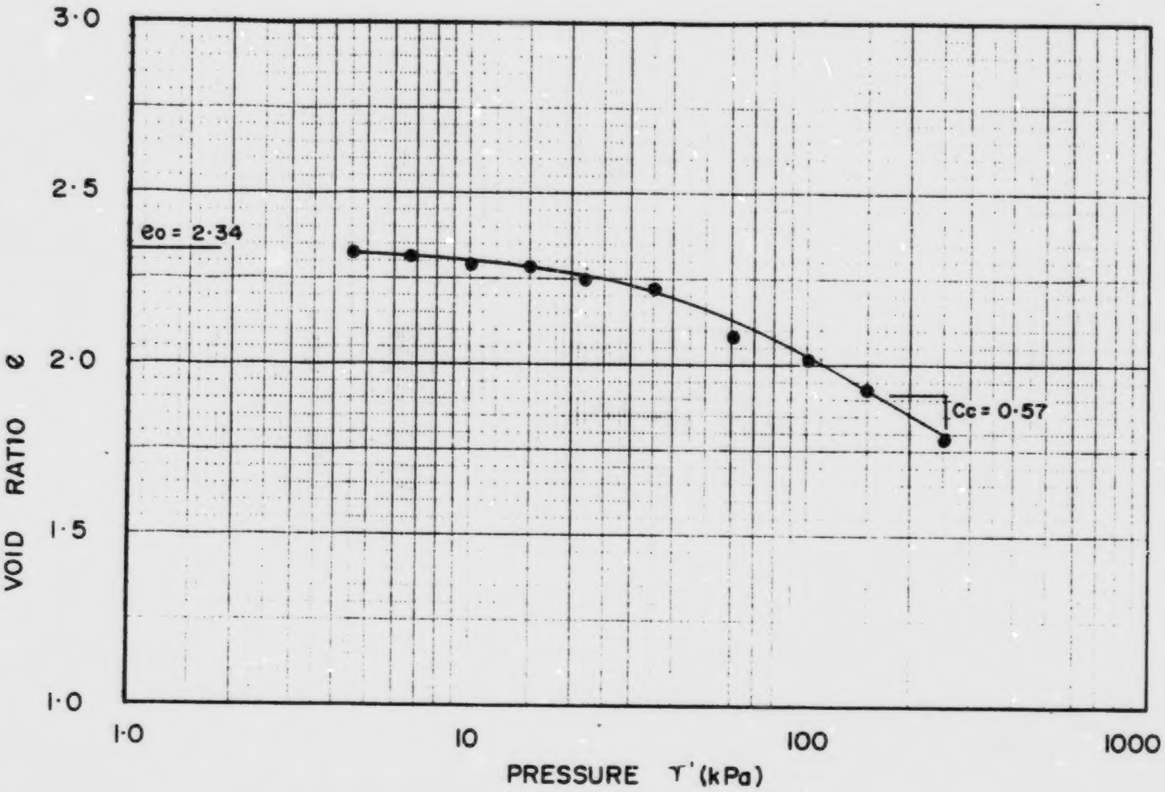
σ' (kPa)	e	C_v (m^2/day)	m_v (kPa^{-1})	k (m / sec)
0	1.43			
		0.21	1.1×10^{-3}	2.6×10^{-8}
4.5	1.42			
		0.42	2.6×10^{-3}	1.2×10^{-7}
6.75	1.41			
		0.42	2.1×10^{-3}	9.7×10^{-8}
10	1.39			
		0.41	1.5×10^{-3}	7.0×10^{-8}
15	1.37			
		0.24	1.1×10^{-3}	3.1×10^{-8}
22	1.35			
		0.62	5.9×10^{-4}	4.1×10^{-8}
35	1.34			
		0.48	5.3×10^{-4}	2.9×10^{-8}
60	1.30			
		0.37	5.1×10^{-4}	2.1×10^{-8}
100	1.26			
		0.57	3.7×10^{-4}	2.4×10^{-8}
150	1.22			
		0.72	2.6×10^{-4}	2.1×10^{-8}
250	1.16			

FIGURE I-3 OEDOMETER TEST RESULTS
BOREHOLE NUTP-S2 Sample 9



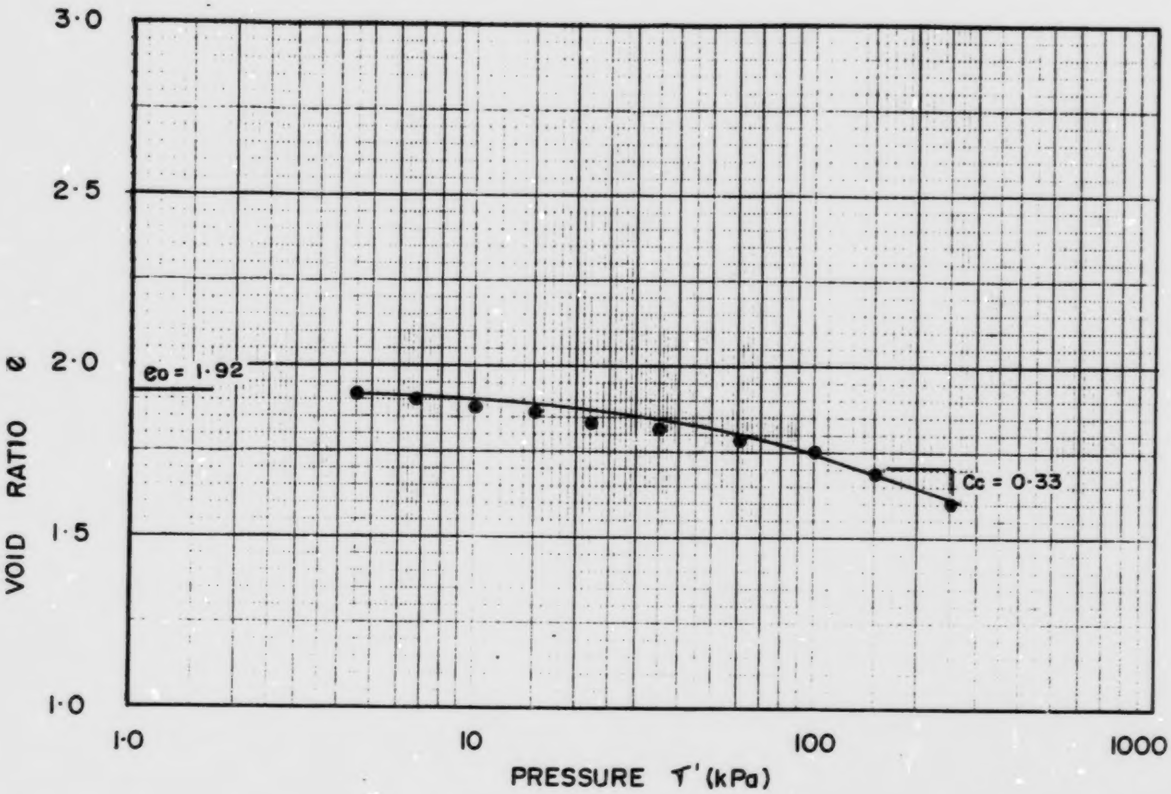
σ' (kPa)	e	C_v (m^2/day)	m_v (kPa^{-1})	k (m / sec)
0	1.70			
		0.05	1.9×10^{-3}	1.1×10^{-8}
4.5	1.68			
		0.65	3.0×10^{-3}	2.2×10^{-7}
6.75	1.66			
		0.07	2.9×10^{-3}	2.3×10^{-8}
10	1.64			
		0.41	1.1×10^{-3}	5.3×10^{-8}
15	1.62			
		0.33	1.9×10^{-3}	6.9×10^{-8}
22	1.59			
		0.33	1.0×10^{-3}	3.9×10^{-8}
35	1.55			
		0.46	8.4×10^{-4}	4.4×10^{-8}
60	1.50			
		0.24	7.2×10^{-4}	2.0×10^{-8}
100	1.43			
		0.47	4.5×10^{-4}	2.4×10^{-8}
150	1.37			
		0.67	3.1×10^{-4}	2.4×10^{-8}
250	1.30			

FIGURE I-4 OEDOMETER TEST RESULTS
BOREHOLE NUTP-S2 Sample 7



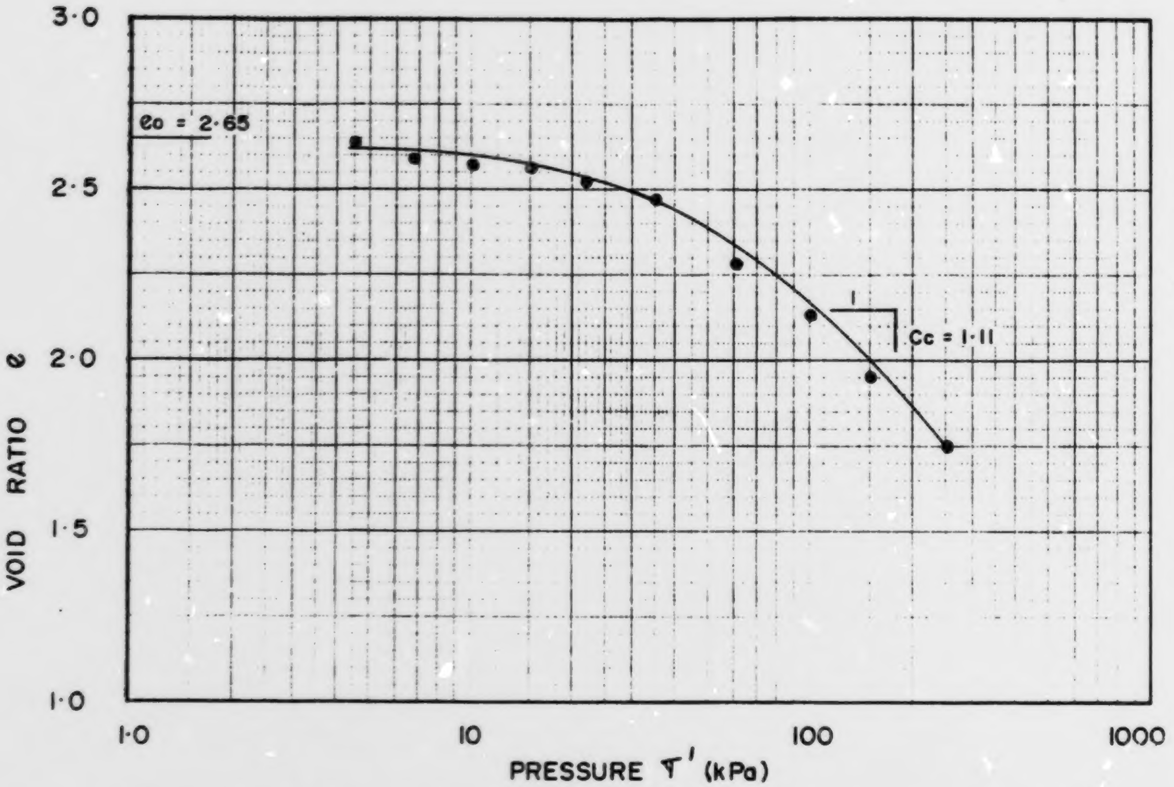
τ' (kPa)	e	C_v (m^2/day)	m_v (kPa^{-1})	k (m / sec)
0	2.34			
		0.43	8.0×10^{-4}	3.9×10^{-8}
4.5	2.33			
		1.17	2.3×10^{-3}	3.0×10^{-7}
6.75	2.31			
		0.26	2.0×10^{-3}	5.8×10^{-8}
10	2.29			
		0.86	8.5×10^{-4}	8.3×10^{-8}
15	2.28			
		0.41	1.3×10^{-3}	6.1×10^{-8}
22	2.25			
		0.96	6.4×10^{-4}	7.0×10^{-8}
35	2.22			
		0.37	1.8×10^{-3}	7.5×10^{-8}
60	2.08			
		0.25	5.3×10^{-4}	1.5×10^{-8}
100	2.01			
		0.23	5.5×10^{-4}	1.4×10^{-8}
150	1.93			
		0.21	4.7×10^{-4}	1.1×10^{-8}
250	1.79			

FIGURE I-5 OEDOMETER TEST RESULTS
BOREHOLE NUTP-S2 Sample 11



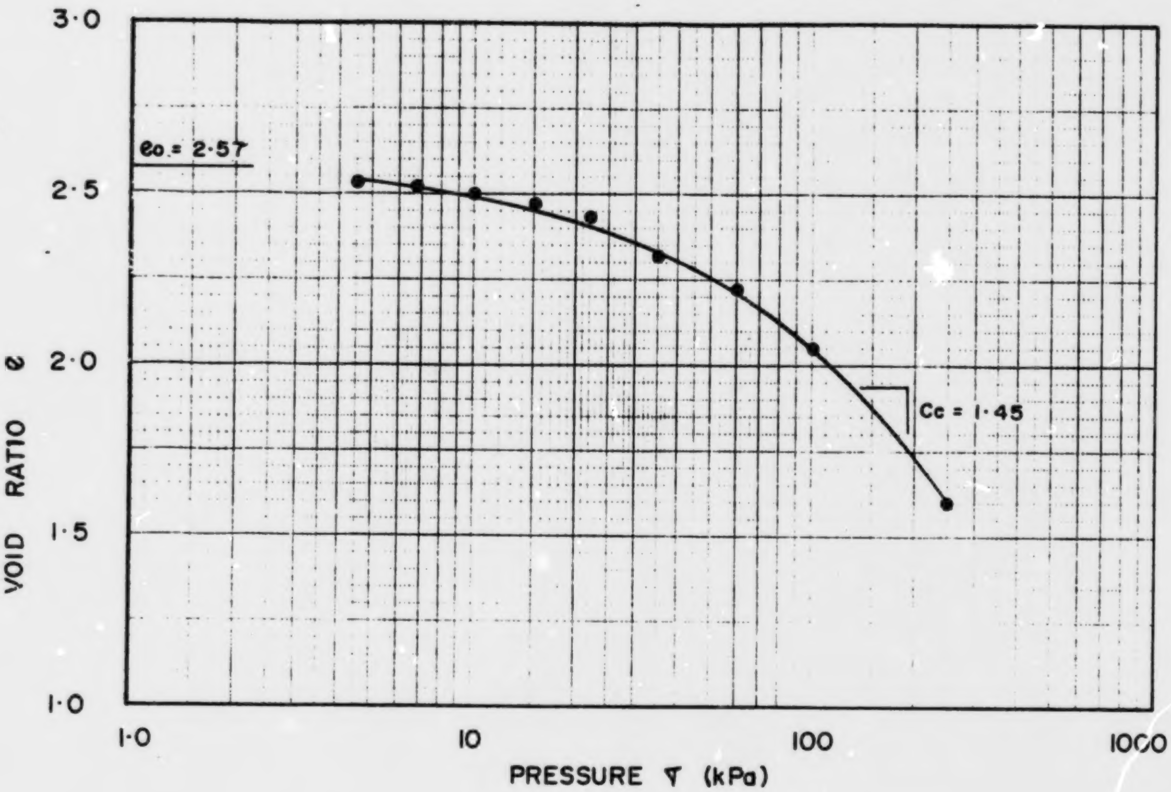
σ' (kPa)	e	C_v (m^2/day)	m_v (kPa^{-1})	k (m / sec)
0	1.92			
		0.43	5.3×10^{-4}	2.6×10^{-8}
4.5	1.91			
		0.66	2.3×10^{-3}	1.7×10^{-7}
6.75	1.90			
		0.52	1.6×10^{-3}	9.4×10^{-8}
10	1.88			
		0.21	1.3×10^{-3}	3.0×10^{-8}
15	1.86			
		0.54	1.1×10^{-3}	6.4×10^{-8}
22	1.84			
		0.63	6.0×10^{-4}	4.3×10^{-8}
35	1.82			
		1.10	3.7×10^{-4}	4.6×10^{-8}
60	1.78			
		0.50	4.5×10^{-4}	2.5×10^{-8}
100	1.75			
		0.31	4.2×10^{-4}	1.5×10^{-8}
150	1.69			
		0.34	3.2×10^{-4}	1.2×10^{-8}
250	1.60			

FIGURE I-6 OEDOMETER TEST RESULTS
BOREHOLE NUTP-S2 Sample 13



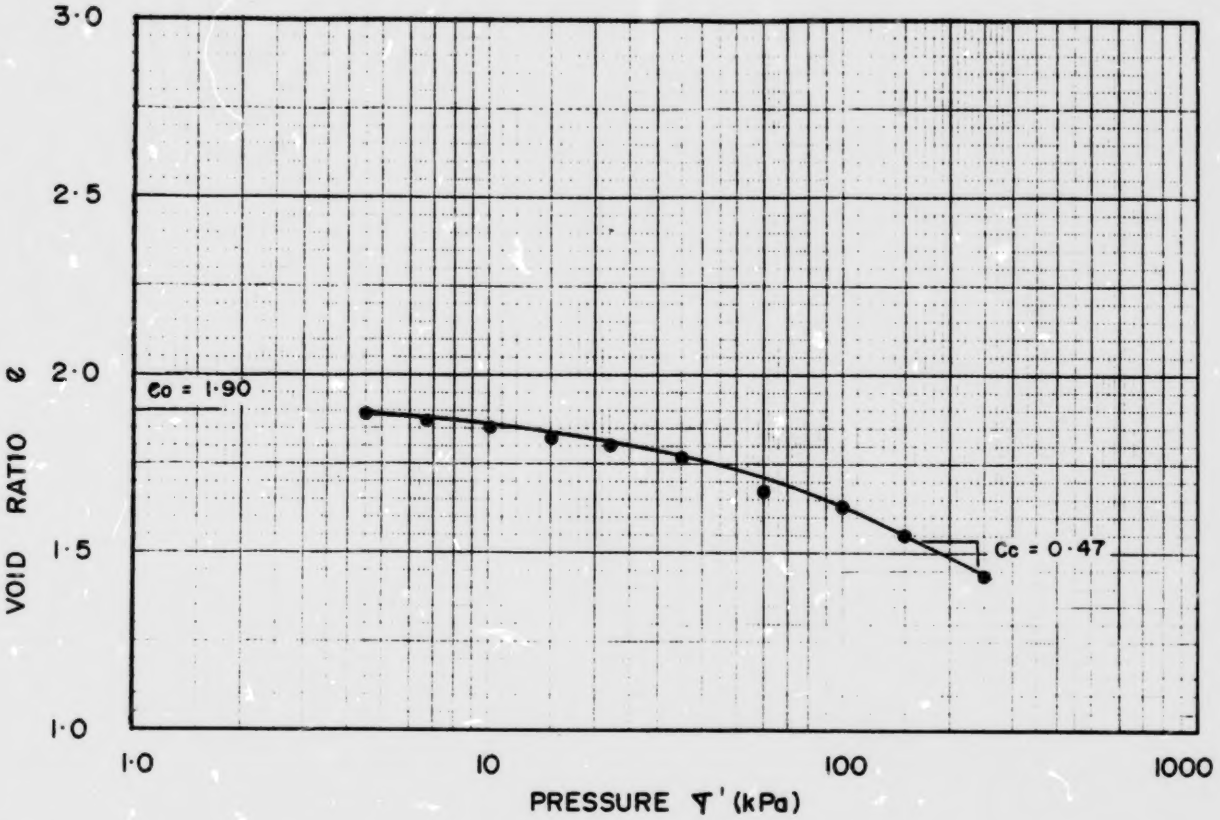
σ' (kPa)	e	C_v (m^2/day)	m_v (kPa^{-1})	k (m / sec)
0	2.65			
		0.52	1.2×10^{-3}	7.2×10^{-8}
4.5	2.63			
		0.05	4.3×10^{-3}	2.4×10^{-8}
6.75	2.59			
		0.21	1.8×10^{-3}	4.4×10^{-8}
10	2.57			
		-	1.3×10^{-3}	-
15	2.56			
		0.33	1.5×10^{-3}	5.7×10^{-8}
22	2.52			
		0.20	1.1×10^{-3}	2.2×10^{-8}
35	2.47			
		0.06	2.2×10^{-3}	1.3×10^{-8}
60	2.28			
		0.34	1.2×10^{-3}	4.4×10^{-8}
100	2.13			
		0.28	1.1×10^{-3}	3.6×10^{-8}
150	1.95			
		0.05	7.0×10^{-4}	4.1×10^{-9}
250	1.75			

FIGURE I-7 OEDOMETER TEST RESULTS
BOREHOLE NUTP-S3 Sample 10



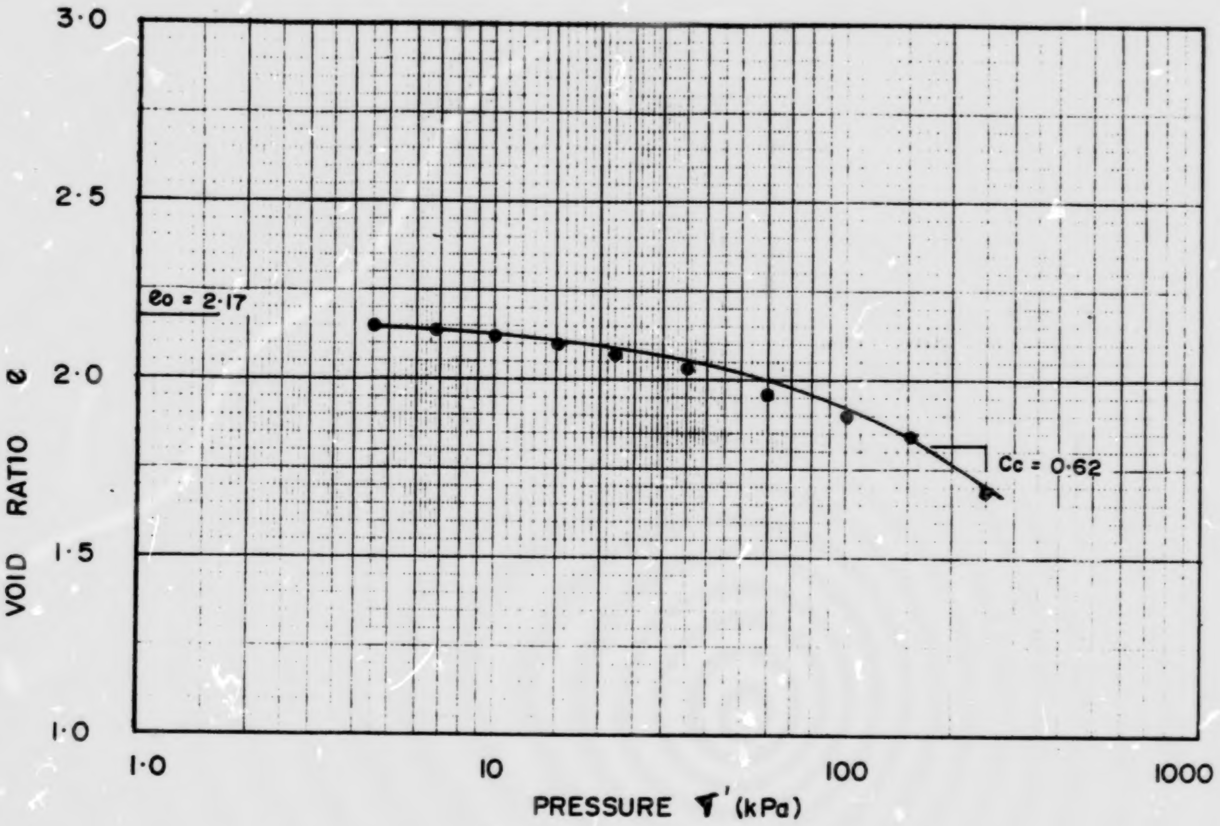
σ' (kPa)	e	C_v (m^2/day)	m_v (kPa^{-1})	k (m / sec)
0	2.57			
		0.19	1.9×10^{-3}	4.1×10^{-8}
4.5	2.54			
		1.15	2.9×10^{-3}	3.8×10^{-7}
6.75	2.52			
		1.14	1.8×10^{-3}	2.3×10^{-7}
10	2.50			
		0.51	1.6×10^{-3}	9.3×10^{-8}
15	2.47			
		0.33	1.9×10^{-3}	7.2×10^{-8}
22	2.43			
		0.12	2.6×10^{-3}	3.6×10^{-8}
35	2.31			
		0.07	1.0×10^{-3}	7.7×10^{-9}
60	2.22			
		0.05	1.4×10^{-3}	7.1×10^{-9}
100	2.05			
		-	-	-
150	-			
		0.12	-	-
250	1.60			

FIGURE I-8 OEDOMETER TEST RESULTS
BOREHOLE NUTP-S3 Sample 14



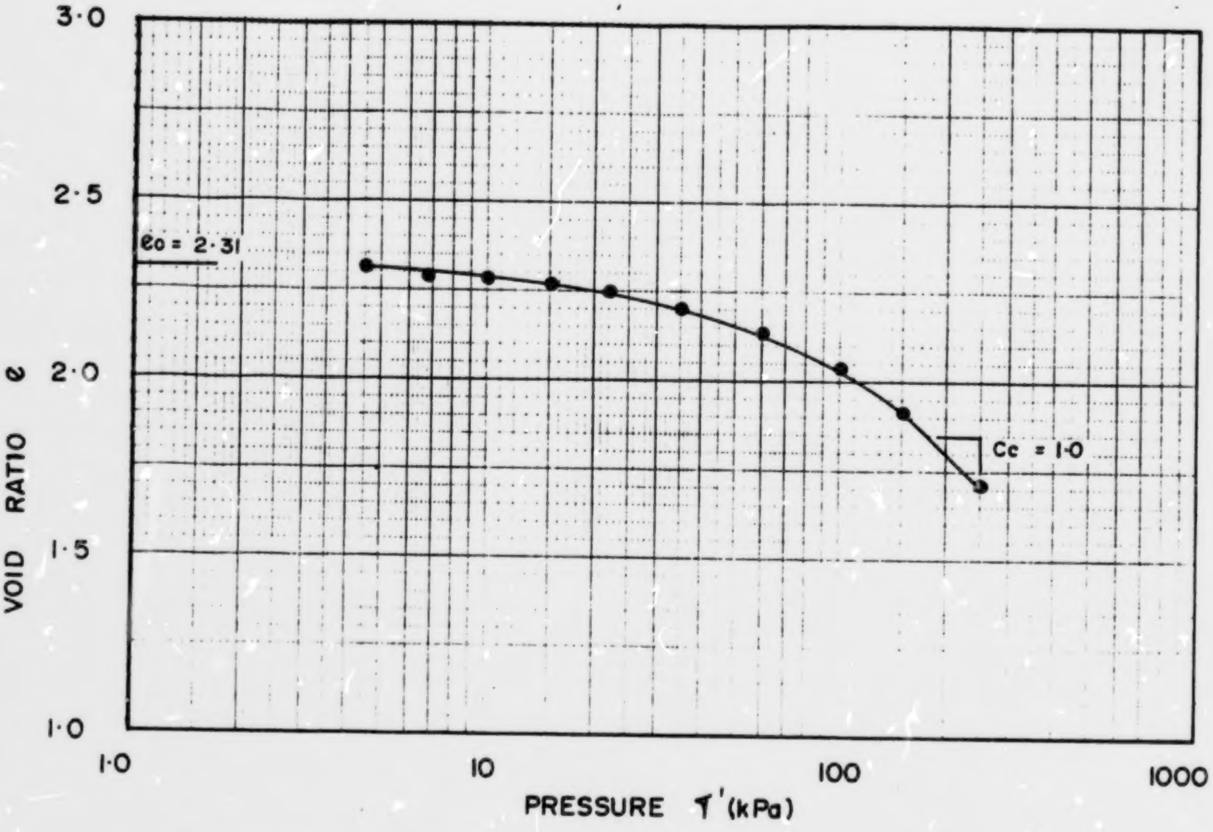
σ' (kPa)	e	C_v (m^2/day)	m_v (kPa^{-1})	k (m / sec)
0	1.90			
		0.14	1.3×10^{-3}	2.1×10^{-8}
4.5	1.89			
		0.12	2.8×10^{-3}	3.8×10^{-8}
6.75	1.87			
		0.06	2.3×10^{-3}	1.5×10^{-8}
10	1.85			
		0.10	1.6×10^{-3}	1.8×10^{-8}
15	1.82			
		0.16	1.2×10^{-3}	2.2×10^{-8}
22	1.80			
		0.15	1.0×10^{-3}	1.8×10^{-8}
35	1.76			
		0.22	1.4×10^{-3}	3.4×10^{-8}
60	1.67			
		0.23	3.3×10^{-4}	8.6×10^{-9}
100	1.63			
		0.13	6.3×10^{-4}	9.3×10^{-9}
150	1.55			
		0.22	4.1×10^{-4}	1.0×10^{-8}
250	1.44			

FIGURE I - 9 OEDOMETER TEST RESULTS
BOREHOLE NUTP-S3 Sample 16



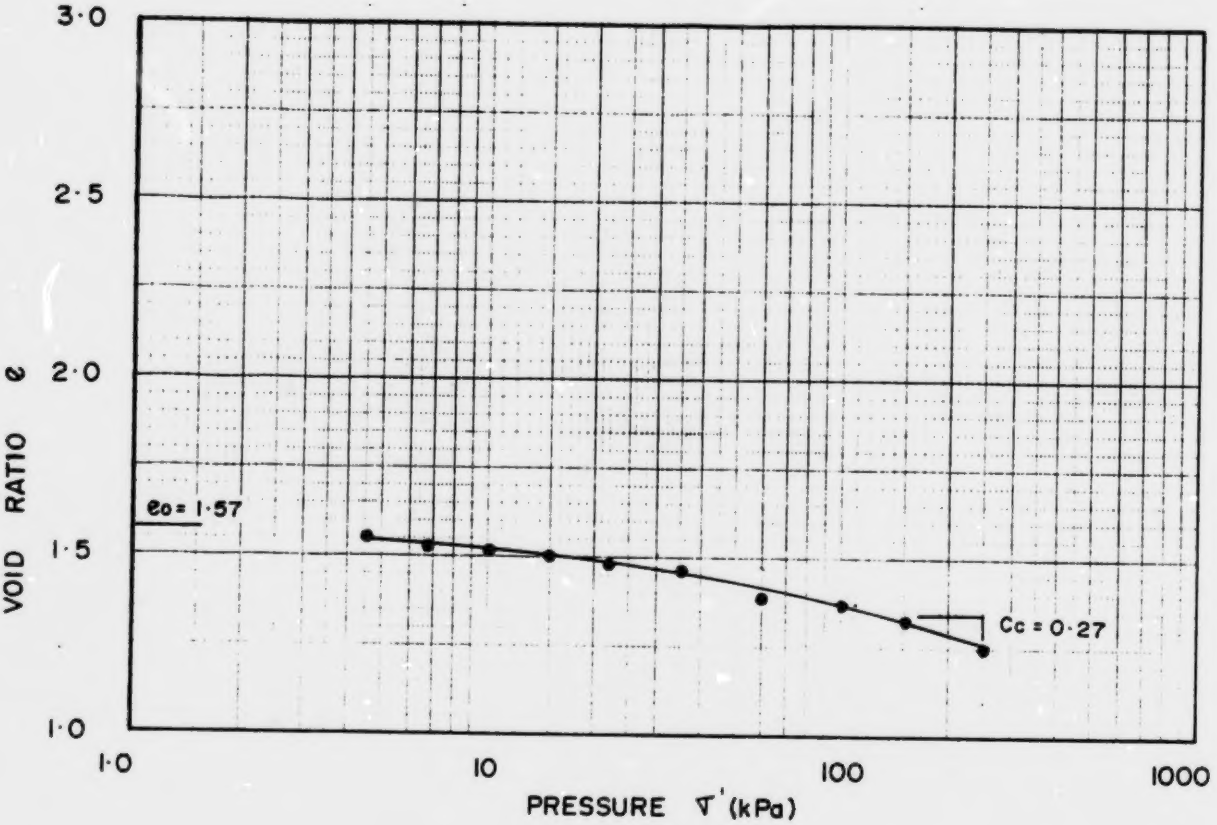
σ' (kPa)	e	c_v (m^2/day)	m_v (kPa^{-1})	k (m / sec)
0	2.17			
		0.13	1.5×10^{-3}	2.2×10^{-8}
4.5	2.15			
		0.07	2.4×10^{-3}	1.9×10^{-8}
6.75	2.14			
		0.09	1.8×10^{-3}	1.8×10^{-8}
10	2.12			
		0.10	1.5×10^{-3}	1.7×10^{-8}
15	2.10			
		0.16	1.4×10^{-3}	2.5×10^{-8}
22	2.07			
		0.20	8.0×10^{-4}	1.8×10^{-8}
35	2.03			
		0.10	9.5×10^{-4}	1.1×10^{-8}
60	1.96			
		0.36	4.9×10^{-4}	2.0×10^{-8}
100	1.90			
		0.23	4.6×10^{-4}	1.2×10^{-8}
150	1.84			
		0.38	5.6×10^{-4}	2.4×10^{-8}
250	1.68			

FIGURE I - 10 OEDOMETER TEST RESULTS
BOREHOLE NUTP-S3 Sample 18



σ' (kPa)	e	C_v (m^2/day)	m_v (kPa^{-1})	k (m / sec)
0	2.31			
		1.77	2.7×10^{-4}	5.4×10^{-8}
4.5	2.31			
		0.43	1.9×10^{-3}	9.2×10^{-8}
6.75	2.29			
		0.35	1.6×10^{-3}	6.3×10^{-8}
10	2.28			
		0.25	1.2×10^{-3}	3.3×10^{-8}
15	2.26			
		0.52	1.8×10^{-4}	1.0×10^{-8}
22	2.25			
		0.40	1.4×10^{-3}	6.2×10^{-8}
35	2.20			
		0.24	8.0×10^{-4}	2.2×10^{-8}
60	2.14			
		0.17	7.2×10^{-4}	1.4×10^{-8}
100	2.04			
		0.07	8.7×10^{-4}	6.9×10^{-9}
150	1.91			
		0.05	6.9×10^{-4}	3.9×10^{-9}
250	1.71			

FIGURE I-11 OEDOMETER TEST RESULTS
BOREHOLE NUTP-S3 Sample 20



σ' (kPa)	e	C_v (m^2/day)	m_v (kPa^{-1})	k (m / sec)
0	1.57			
		0.07	2.5×10^{-3}	2.0×10^{-8}
4.5	1.55			
		0.26	3.0×10^{-3}	8.8×10^{-8}
6.75	1.53			
		0.10	1.8×10^{-3}	2.1×10^{-8}
10	1.51			
		0.36	8.0×10^{-4}	3.2×10^{-8}
15	1.50			
		0.18	1.1×10^{-3}	2.2×10^{-8}
22	1.48			
		0.27	7.1×10^{-4}	2.2×10^{-8}
35	1.46			
		0.59	1.1×10^{-3}	7.5×10^{-8}
60	1.39			
		0.22	1.9×10^{-4}	4.7×10^{-9}
100	1.37			
		0.22	4.0×10^{-4}	1.0×10^{-8}
150	1.33			
		0.28	3.2×10^{-4}	1.0×10^{-8}
250	1.25			

FIGURE 1-12 OEDOMETER TEST RESULTS
BOREHOLE NUTP-S3 Sample 22

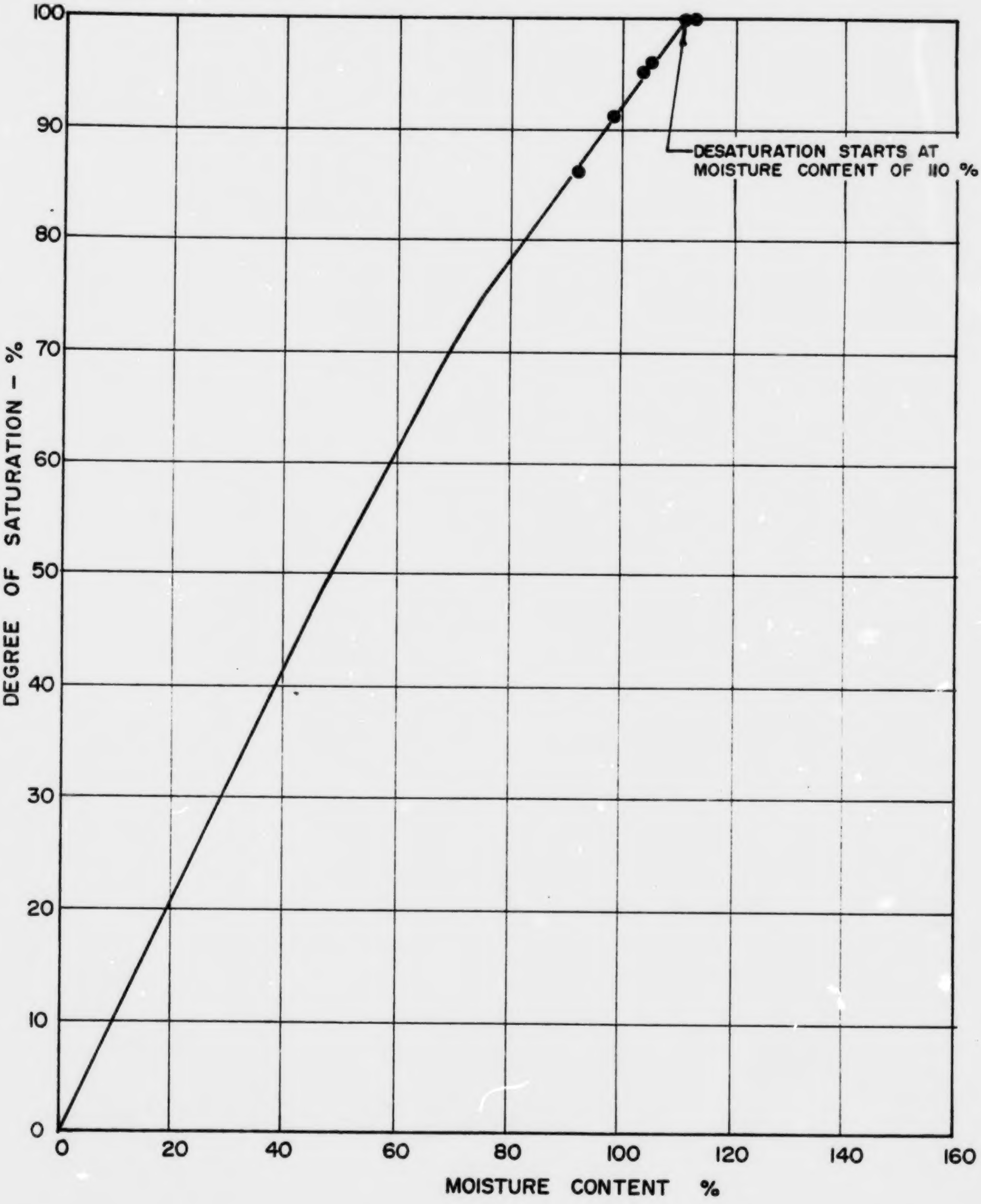


FIGURE I - 13 DEGREE OF SATURATION VS. MOISTURE CONTENT
PRESSURE MEMBRANE TEST RESULTS
BOREHOLE NUTP - S2 SAMPLE NO. 4

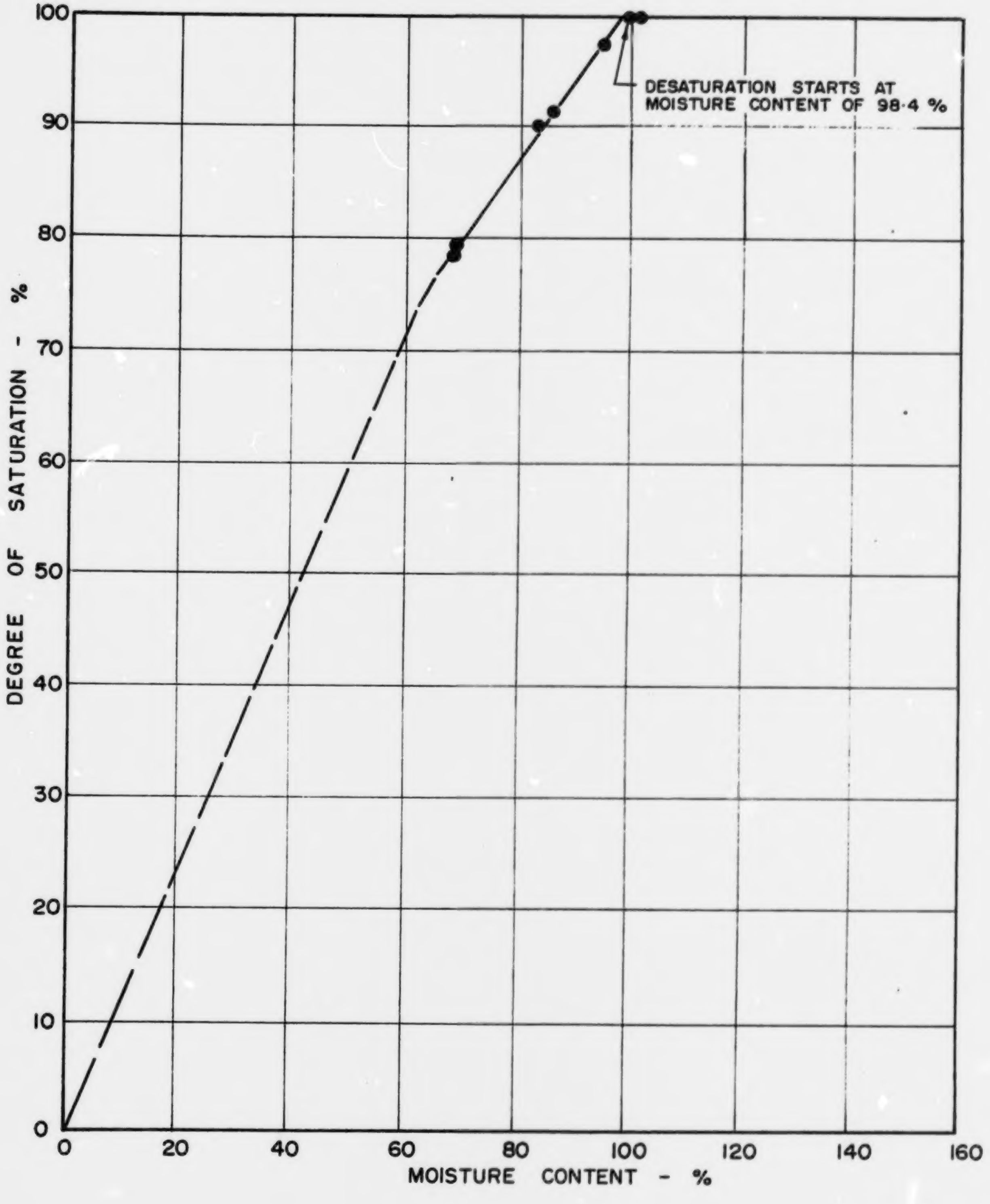


FIGURE I - 14

DEGREE OF SATURATION VS. MOISTURE CONTEN
PRESSURE MEMBRANE TEST RESULTS
BOREHOLE NUTP - S3 SAMPLE NO. 13

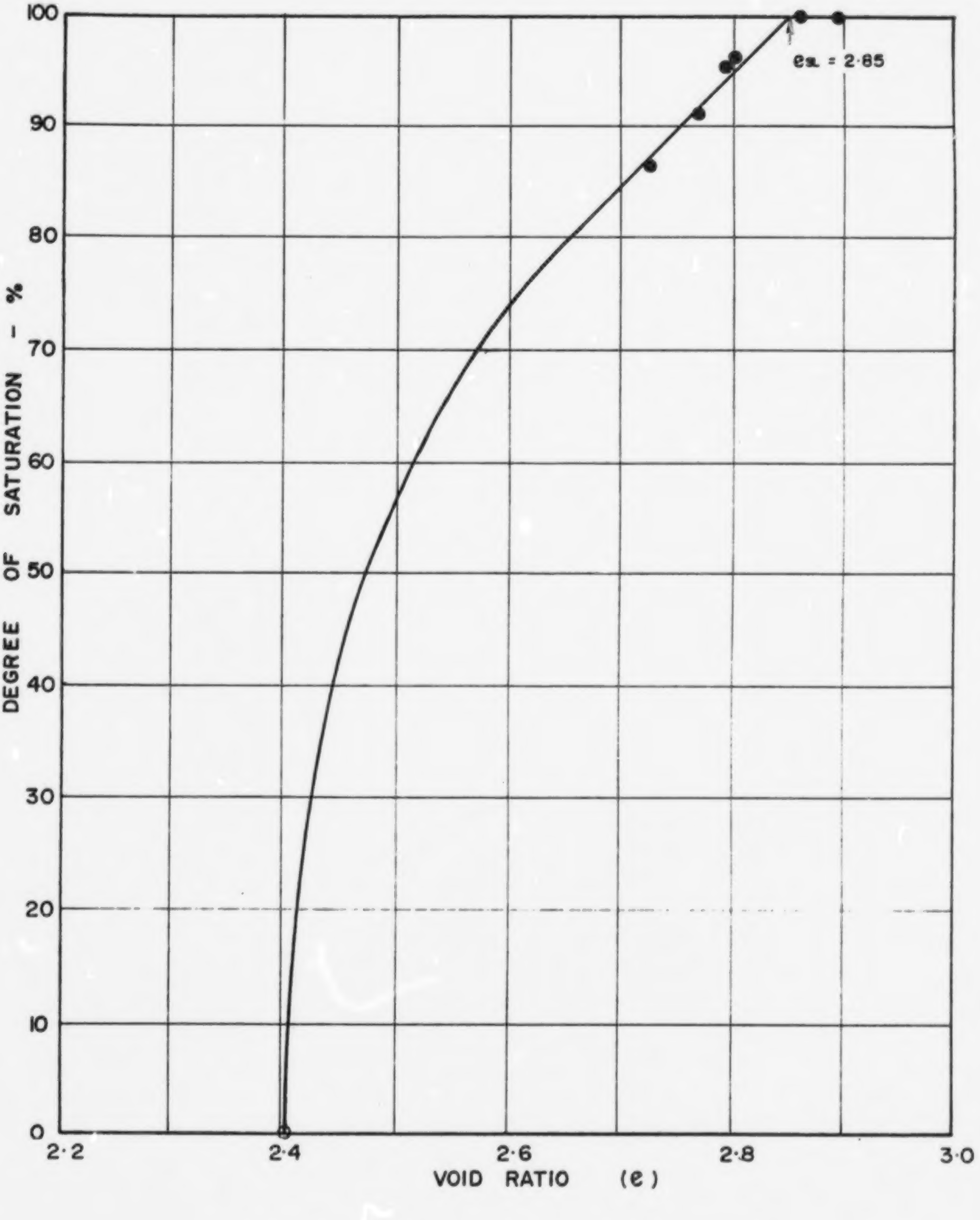


FIGURE I - 15 DEGREE OF SATURATION VS. VOID RATIO
PRESSURE MEMBRANE TEST RESULTS
BOREHOLE NUTP - S2 SAMPLE NO. 4

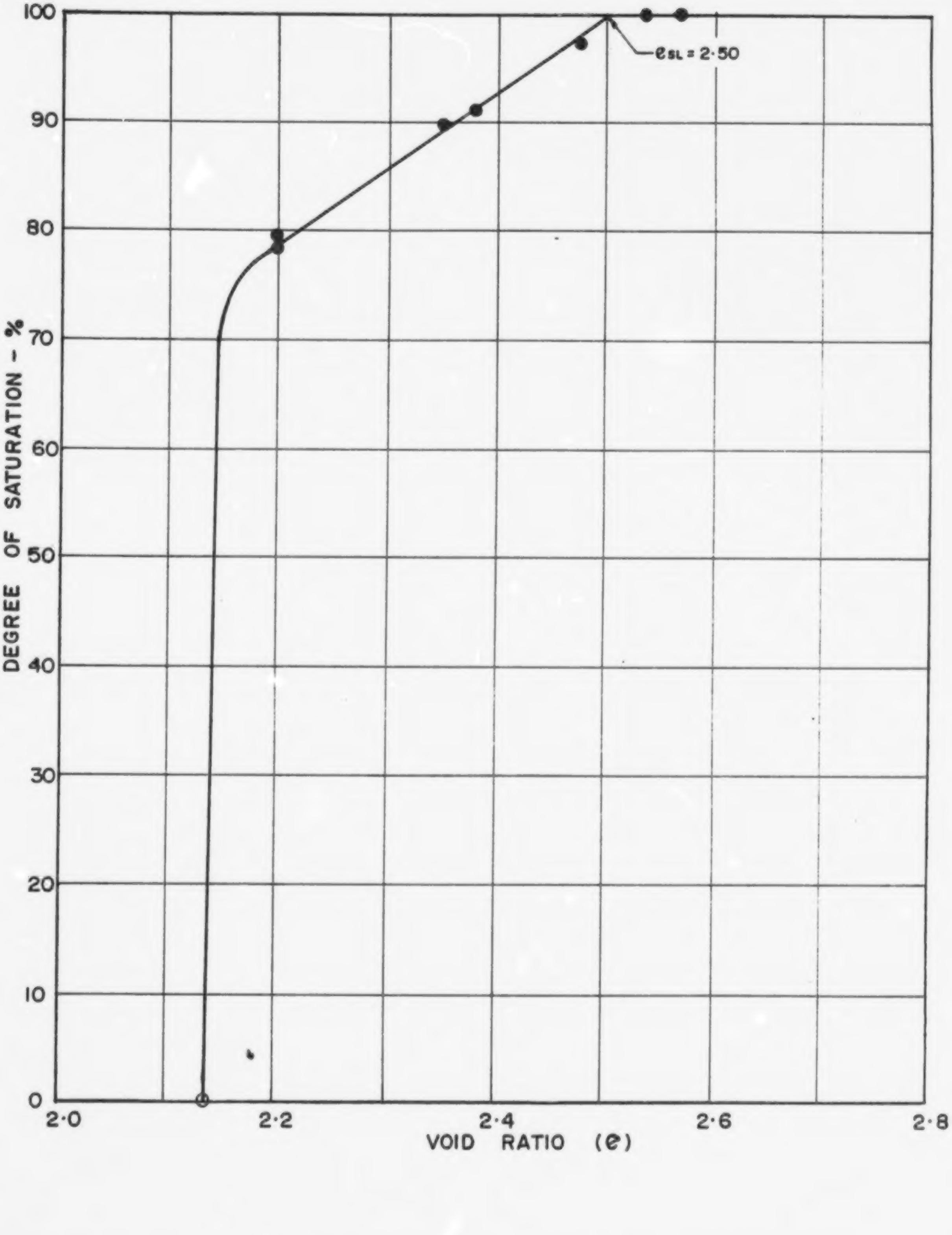


FIGURE I - 16

DEGREE OF SATURATION VS. VOID RATIO
PRESSURE MEMBRANE TEST RESULTS
BOREHOLE NUTP - S3 SAMPLE NO. 13 .

APPENDIX II

PIEZOCONE TEST RESULTS AND INTERPRETATION

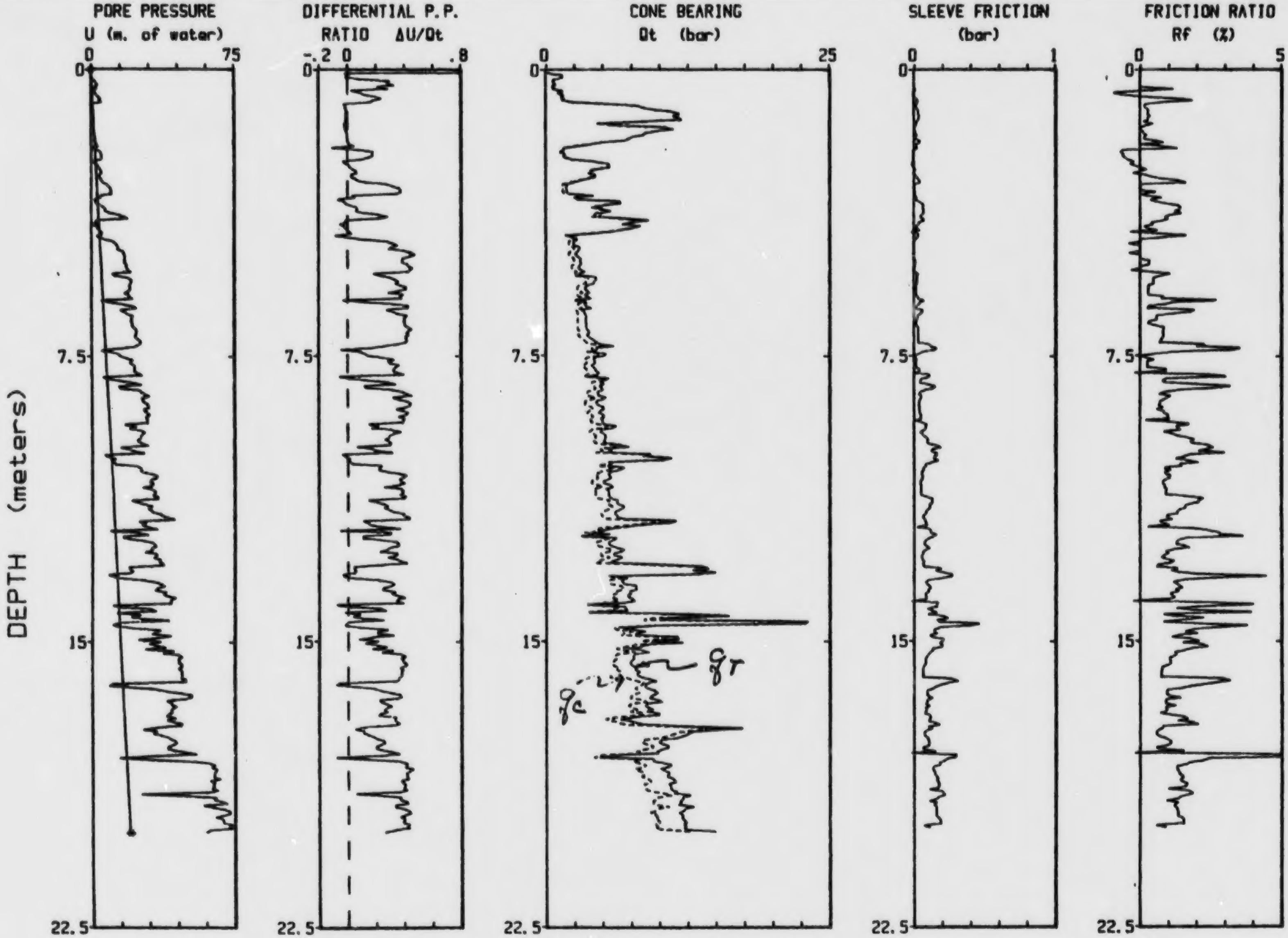
by Dr. R.G. Campanella
University of British Columbia

RABBIT LAKE STUDY

Site Location: ELDOR MINES
On Site Loc: Hole 10/A

CPT Date : 86/02/09
Cone Used: FOUNDEX-3015

Page No: 1 / 1
Comments: Not Pre-bored



Depth Increment : .05 m

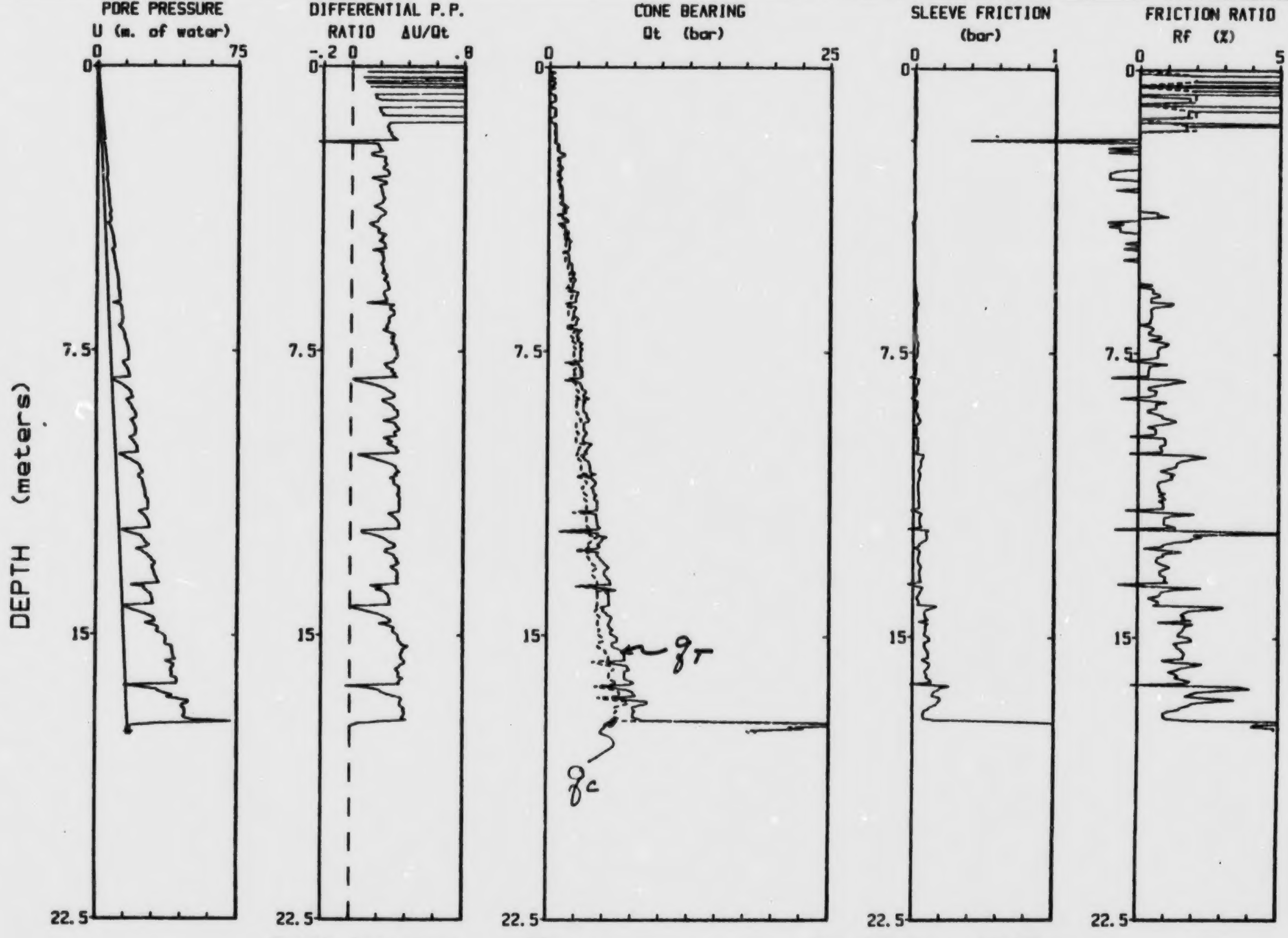
Max Depth : 20 m

RABBIT LAKE STUDY

Site Location: ELDOR MINES
On Site Loc: Hole 2

CPT Date : 86/02/10
Cone Used: FOUNDEX-3015

Page No: 1 / 1
Comments: Not Pre-bored



Depth Increment : .05 m

Max Depth : 17.55 m

RABBIT LAKE STUDY

Site :ELDOR MINES
On Site Loc:Hole 1-1A
Comments :Combined
Tot. Unit Wt. (avg) : 17 kN/m^3

CPT Date :86/02/09
Cone Used :FOUNDEX-3015
Water table (meters) : 0

DEPTH (meters)	(feet)	Qc (avg) (bar)	Fs (avg) (bar)	Rf (avg) (%)	SIGV' (kPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	Su kPa
0.50	1.64	0.75	0.00	0.13	1.80	undefined	UNDFND	UNDFD	UDF	UNDEFINED
1.00	3.28	3.40	0.01	0.26	5.39	sensitive fine grained	UNDFND	UNDFD	2	29.7
1.50	4.92	9.75	0.02	0.24	8.98	sandy silt to clayey silt	UNDFND	UNDFD	4	86.7
2.00	6.56	7.08	0.02	0.23	12.58	sensitive fine grained	UNDFND	UNDFD	4	61.6
2.50	8.20	2.65	-0.01	-0.30	16.17	undefined	UNDFND	UNDFD	UDF	UNDEFINED
3.00	9.84	3.58	0.01	0.39	19.76	sensitive fine grained	UNDFND	UNDFD	2	28.2
3.50	11.48	3.00	0.01	0.50	23.36	sensitive fine grained	UNDFND	UNDFD	2	22.2
4.00	13.12	5.50	0.06	1.09	26.95	sensitive fine grained	UNDFND	UNDFD	3	44.2
4.50	14.76	4.70	0.02	0.34	30.55	sensitive fine grained	UNDFND	UNDFD	2	36.1
5.00	16.40	2.25	-0.00	-0.04	34.14	undefined	UNDFND	UNDFD	UDF	UNDEFINED
5.50	18.04	2.73	0.01	0.40	37.73	sensitive fine grained	UNDFND	UNDFD	1	16.6
6.00	19.69	3.08	0.02	0.52	41.33	sensitive fine grained	UNDFND	UNDFD	2	19.0
6.50	21.33	2.83	0.04	1.27	44.92	sensitive fine grained	UNDFND	UNDFD	1	16.0
7.00	22.97	2.87	0.02	0.84	48.51	sensitive fine grained	UNDFND	UNDFD	1	15.6
7.50	24.61	4.07	0.07	1.72	52.11	sensitive fine grained	UNDFND	UNDFD	2	25.7
8.00	26.25	3.55	0.02	0.70	55.70	sensitive fine grained	UNDFND	UNDFD	2	20.2
8.50	27.89	4.15	0.08	1.83	59.29	sensitive fine grained	UNDFND	UNDFD	2	24.9
9.00	29.53	3.81	0.04	1.00	62.89	sensitive fine grained	UNDFND	UNDFD	2	21.1
9.50	31.17	4.31	0.06	1.35	66.48	sensitive fine grained	UNDFND	UNDFD	2	24.8
10.00	32.81	4.83	0.11	2.30	70.07	silty clay to clay	UNDFND	UNDFD	3	28.8
10.50	34.45	6.94	0.12	1.70	73.67	clayey silt to silty clay	UNDFND	UNDFD	3	47.2
11.00	36.09	4.45	0.05	1.12	77.26	sensitive fine grained	UNDFND	UNDFD	2	23.8
11.50	37.73	5.03	0.10	1.91	80.86	silty clay to clay	UNDFND	UNDFD	3	28.3
12.00	39.37	6.54	0.07	1.12	84.45	sensitive fine grained	UNDFND	UNDFD	3	41.2
12.50	41.01	4.43	0.10	2.37	88.04	clay	UNDFND	UNDFD	4	21.3
13.00	42.65	5.47	0.07	1.21	91.64	sensitive fine grained	UNDFND	UNDFD	3	30.0
13.50	44.29	9.58	0.16	1.69	95.23	clayey silt to silty clay	UNDFND	UNDFD	5	66.6
14.00	45.93	6.22	0.09	1.46	98.82	clayey silt to silty clay	UNDFND	UNDFD	3	35.2
14.50	47.57	9.37	0.16	1.71	102.42	clayey silt to silty clay	UNDFND	UNDFD	5	63.1
15.00	49.21	9.53	0.21	2.23	106.01	clayey silt to silty clay	UNDFND	UNDFD	5	63.8
15.50	50.85	7.14	0.13	1.82	109.60	clayey silt to silty clay	UNDFND	UNDFD	4	41.3
16.00	52.49	6.40	0.09	1.34	113.20	sensitive fine grained	UNDFND	UNDFD	3	33.8
16.50	54.13	8.00	0.15	1.90	116.79	clayey silt to silty clay	UNDFND	UNDFD	4	47.6
17.00	55.77	7.86	0.09	1.17	120.39	clayey silt to silty clay	UNDFND	UNDFD	4	45.5
17.50	57.41	9.97	0.11	1.08	123.98	clayey silt to silty clay	UNDFND	UNDFD	5	63.9
18.00	59.06	8.42	0.11	1.28	127.57	clayey silt to silty clay	UNDFND	UNDFD	4	49.1
18.50	60.70	7.92	0.18	2.27	131.17	silty clay to clay	UNDFND	UNDFD	5	43.7
19.00	62.34	9.06	0.17	1.89	134.76	clayey silt to silty clay	UNDFND	UNDFD	5	53.3

Dr - All sands (Jamiolkowski et al. 1985) PHI - Robertson and Campanella 1983 Su: Nk= 11

** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.04) ****

RABBIT LAKE STUDY

```
Site          :ELDOR MINES          On Site Loc:Hole 1-1A                 Page No. 2
--------------------------------------------------------------------------------------------------------------------------
     DEPTH          Qc (avg)   Fs (avg)   Rf (avg)    SIGV'       SOIL BEHAVIOUR TYPE        Eq - Dr   PHI    SPT     Su
(  ters)  (feet)     (bar)      (bar)      (%)       (kPa)                                    (%)     deg.    N      kPa
--------------------------------------------------------------------------------------------------------------------------
 19.50    63.98      9.87       0.16       1.62     138.35   clayey silt to silty clay     UNDFND   UNDFD    5      59.9
 20.00    65.62     10.11       0.17       1.63     141.95   clayey silt to silty clay     UNDFND   UNDFD    5      61.3
--------------------------------------------------------------------------------------------------------------------------

    Dr -  All sands (Jamiolkowski et al. 1985)      PHI -     Robertson and Campanella 1983        Su: Nk= 11

**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.04) ****
```

RABBIT LAKE STUDY

Site :ELDOR MINES
On Site Loc:Hole 2
Comments :Not Pre-bored
Tot. Unit Wt. (avg) : 17 kN/m^3

CPT Date :86/02/10
Cone Used :FOUNDEX-3015
Water table (meters) : 0

DEPTH (meters)	(feet)	Qc (avg) (bar)	Fs (avg) (bar)	Rf (avg) (%)	SIGV' (kPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	Su kPa
0.50	1.64	0.20	0.01	3.00	1.80	undefined	UNDFND	UNDFD	UDF	UNDEFINED
1.00	3.28	0.25	0.01	2.40	5.39	undefined	UNDFND	UNDFD	UDF	UNDEFINED
1.50	4.92	0.30	0.01	2.33	8.98	undefined	UNDFND	UNDFD	UDF	UNDEFINED
2.00	6.56	0.50	-0.00	-0.20	12.58	undefined	UNDFND	UNDFD	UDF	UNDEFINED
2.50	8.20	0.85	-0.00	-0.24	16.17	undefined	UNDFND	UNDFD	UDF	UNDEFINED
3.00	9.84	0.82	-0.00	-0.61	19.76	undefined	UNDFND	UNDFD	UDF	UNDEFINED
3.50	11.48	0.95	-0.00	-0.11	23.36	undefined	UNDFND	UNDFD	UDF	UNDEFINED
4.00	13.12	1.15	0.00	0.26	26.95	sensitive fine grained	UNDFND	UNDFD	1	4.6
4.50	14.76	1.33	-0.01	-0.53	30.55	undefined	UNDFND	UNDFD	UDF	UNDEFINED
5.00	16.40	1.50	-0.00	-0.13	34.14	undefined	UNDFND	UNDFD	UDF	UNDEFINED
5.50	18.04	1.70	-0.00	-0.06	37.73	undefined	UNDFND	UNDFD	UDF	UNDEFINED
6.00	19.69	1.88	0.01	0.43	41.33	sensitive fine grained	UNDFND	UNDFD	1	6.1
6.50	21.33	1.85	0.01	0.70	44.92	sensitive fine grained	UNDFND	UNDFD	1	7.1
7.00	22.97	2.00	0.01	0.45	48.51	sensitive fine grained	UNDFND	UNDFD	1	7.7
7.50	24.61	2.23	0.02	0.94	52.11	sensitive fine grained	UNDFND	UNDFD	1	9.0
8.00	26.25	2.23	0.01	0.49	55.70	sensitive fine grained	UNDFND	UNDFD	1	8.2
8.50	27.89	2.33	0.01	0.65	59.29	sensitive fine grained	UNDFND	UNDFD	1	8.3
9.00	29.53	2.63	0.01	0.53	62.89	sensitive fine grained	UNDFND	UNDFD	1	10.3
9.50	31.17	2.68	0.03	1.01	66.48	sensitive fine grained	UNDFND	UNDFD	1	10.0
10.00	32.81	2.68	0.02	0.86	70.07	sensitive fine grained	UNDFND	UNDFD	1	9.2
10.50	34.45	2.78	0.05	1.62	73.67	sensitive fine grained	UNDFND	UNDFD	1	9.3
11.00	36.09	3.15	0.04	1.21	77.26	sensitive fine grained	UNDFND	UNDFD	2	12.0
11.50	37.73	3.30	0.03	1.06	80.86	sensitive fine grained	UNDFND	UNDFD	2	12.6
12.00	39.37	3.38	0.04	1.07	84.45	sensitive fine grained	UNDFND	UNDFD	2	12.5
12.50	41.01	3.55	0.07	2.00	88.04	clay	UNDFND	UNDFD	4	13.3
13.00	42.65	3.53	0.04	0.99	91.64	sensitive fine grained	UNDFND	UNDFD	2	12.3
13.50	44.29	4.15	0.05	1.13	95.23	sensitive fine grained	UNDFND	UNDFD	2	17.2
14.00	45.93	4.33	0.03	0.79	98.82	sensitive fine grained	UNDFND	UNDFD	2	18.0
14.50	47.57	4.47	0.08	1.88	102.42	silty clay to clay	UNDFND	UNDFD	3	18.6
15.00	49.21	4.58	0.09	1.95	106.01	silty clay to clay	UNDFND	UNDFD	3	18.7
15.50	50.85	4.95	0.10	2.02	109.60	silty clay to clay	UNDFND	UNDFD	3	21.4
16.00	52.49	5.38	0.10	1.82	113.20	silty clay to clay	UNDFND	UNDFD	4	24.5
16.50	54.13	5.70	0.14	2.39	116.79	silty clay to clay	UNDFND	UNDFD	4	26.7
17.00	55.77	6.25	0.14	2.26	120.39	silty clay to clay	UNDFND	UNDFD	4	30.9
17.50	57.41	14.48	0.56	3.89	123.98	silty clay to clay	UNDFND	UNDFD	10	104.9

Dr - All sands (Jamiolkowski et al. 1985) PHI - Robertson and Campanella 1983 Su: Nk= 11

**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.04) ****

APPENDIX III

MINERALOGICAL ANALYSES

by Dr. R.G. Arnold
Saskatchewan Research Council

MINERALOGICAL ANALYSIS OF TWO RABBIT LAKE TAILINGS SAMPLES
SUBMITTED BY GEOCON

by

R.G. Arnold

SRC Publication No. R-855-4-C-86

April, 1986

MINERALOGICAL ANALYSIS OF TWO RABBIT LAKE TAILINGS SAMPLES SUBMITTED BY GEOCON

Introduction:

Two tailings samples, designated No.43139, hole S-2, #2 & #12, were received on or about March 10, 1986.

SRC was requested to investigate the mineralogy, particle size and quantity of gypsum and clay minerals in the two tailings samples by Mr. W. F. Tao in a letter to R. G. Arnold dated February 28, 1986.

X-ray Diffraction Analysis

X-ray diffraction patterns were made of portions of each sample. A few milligrams of sample was ground dry in a mortar and packed dry into a sample holder.

The diffraction pattern of sample #12 (Fig. 1) showed the presence of clay (trichlorite), quartz and gypsum ($CaSO_4 \cdot 2H_2O$).

The diffraction pattern of sample #2 (Fig. 2) showed the presence of clay (trichlorite), quartz and bassanite ($CaSO_4 \cdot 0.5H_2O$).

Re-running sample #2 on a smear-slide using a few drops of distilled water to make the material adhere to the slide, resulted in a diffraction pattern consisting of clay, quartz and only gypsum (Fig. 3).

Evidently, bassanite had converted to gypsum in just a few minutes after wetting the sample.

Re-running sample #12 on a smear slide using a similar procedure of wetting the sample showed no appreciable change (Fig.4).

Visual Examination

Sample #2 was not obviously layered, and was light brown to off-white in color, and under the microscope was found to consist of a fine grained mixture of clay, quartz, minor opaque minerals including yellow to red grains (iron oxide?). The sample contained well developed small white, translucent needles, laths and plates of bassanite.

Sample #12 was rusty brown in color and obviously layered, with layers ranging in color from light brown to red. Under the microscope, the sample was composed of a fine grained mixture of clay, quartz, minor opaque minerals, and small white, translucent needles, laths and plates of gypsum, which were poorly developed relative to those in sample #2.

Scanning Electron Microscope (SEM) Examination

Four SEM photgraphs were taken of each sample to characterize the particle sizes and shapes, and provide an indication of the texture and structure of the samples (see Figs. 5 and 6).

Sample #2

Photo 0524: disaggregated material consisting of well developed needles, laths and plates of bassanite associated with clay minerals and quartz. The larger equidimensional grains may be quartz buried in clay minerals. The fine grained material occuring as clumps, masses and coatings on other minerals are the clay minerals. The largest bassanite lath/needle measures 165 X 20 um; the largest bassanite plate is 85 X 70 um (micrometre or micron).

Photo 0525: surface of a tailings fragment showing the texture and structure. The photo shows bassanite laths and plates with abundant clay mineral debris. Quartz grains are probably covered by clay minerals.The largest bassinite lath/needle is 48 X 5 um; and the largest bassanite plate is 30 X 18 um.

Photo 0526: similar to 0525 but the photo is of a different fragment. The largest bassanite lath/needle is 30 X 12 um.

Photo 0527: disaggregated material. Description is similar to 0524. The bassanite gypsum lath/needle is 90 X 20 um; the largest bassanite plate is 110 X 100 um.

Sample #12

Photo 0528: disaggregated material showng needles and laths of gypsum coated with clay, and abundant clumps and masses of clay. Quartz grains are probably the more equidimensional grains, but are buried in clay. The largest gypsum lath/needle is 209 X 27 um.

Photo 0529: surface texture and structure of a tailings fragment showing laths and a few plates of gypsum and abundant flakes, clumps and masses of clay. Quartz grains are probably present but are obscured by clay minerals. The largest gypsum lath is 100 X 10 um.

Photo 0530: surface texture and structure of another tailings fragment. Description is similar to 0529 but the proportion of clay is greater. In fact, few gypsum needles/laths are present in this view.

Photo 0531: surface texture and structure of another fragment showing primarily needles and a few laths of gypsum associated with abundant clay debris. The largest gypsum lath is 150 X 18 um, and the largest needle is 46 X 4 um.

Quantitative Mineralogy

The quantities of gypsum in sample #12 and bassanite in sample #2 were estimated by dissolving them from weighed samples with hot water over a period of 5 days. The dissolved calcium sulfate was then recovered by evaporation (at moderate temperature) of the leach solution and weighed. An x-ray diffraction pattern of the undissolved residues indicated that gypsum and bassanite had been quantitatively removed to below the limit of detectability (approx. 2 to 3 %).

An x-ray diffraction pattern of the residue recovered from sample #2 by evaporation of the leach solution, indicated that only bassanite had formed during the evaporation process. An examination of the bassanite residue under the microscope showed that very well formed euhedral white,transparent needles, laths and plates had developed, such as the best of those shown in Fig. 5.

An X-ray diffraction pattern (Fig. 7) of the residue from the evaporation of leach water from sample #12, showed that a mixture of gypsum and bassanite had formed.

The percentages of water soluble calcium sulfate are as follows:

Sample No.	% Gypsum	% Bassanite
#2		19.0
#12	17.5	

The quantity of gypsum and bassinite were also estimated from the acid soluble calcium in the two samples. The results are as follows:

Sample No.	% Gypsum	% Bassanite
#2		18.4
#12	14.2	

The estimates of gypsum and bassanite in the samples obtained by the two methods agree reasonably well.

The quantity of trichlorite was estimated from the weight of the minus 5 um (micron) material extracted from the two samples by a sedimentation method. The determinations were done on residues of the samples after removal of gypsum and bassanite. The results were then recalculated to the original sample weight before removal of gypsum and bassanite.

Sample No.	% Trichlorite
#2	24.7
#12	24.9

Discussion

Samples #2 and #12 are similar in terms of the type and content of clay, quartz and the quantity of calcium sulfate they contain; however, the two samples differ in their texture and structure and the type of calcium sulfate they contain. Sample #2 contains bassanite and sample #12 contains gypsum.

The difference in texture and structure of the two samples as seen in SEM photos may be due in part to the difference in crystal development of calcium sulfate. Sample #2 shows abundant well developed bassanite needles, blades and plates; whereas, sample #12 contains smaller and less well developed needles and laths of gypsum.

The conditions of formation of gypsum and bassanite are not clear, but the chemical relationship between the two minerals is expressed by the following equation:

$$\underset{\text{bassanite}}{2[CaSO_4 \cdot 0.5H_2O]} + 3H_2O \rightleftharpoons \underset{\text{gypsum}}{2[CaSO_4 \cdot 2H_2O]}$$

The conversion of bassanite to gypsum by the addition of water is very rapid; however, the conversion of gypsum to bassanite under atmospheric conditions is not supposed to occur. Consequently, the development of bassanite in atmospheric conditions is believd to occur directly from a calcium sulfate rich solution, and not by the dissociation of gypsum into bassanite and water as suggested by the equation above.

Bassanite has been found in Australia in naturally deposited sediments millions of years old interbedded with gypsum bearing beds in an arid environment. It therefore is able to form under atmospheric conditions given the proper physical/chemical environment.

The evaporation of the leach solutions, at moderately elevated temperatures, in the laboratory (described above) resulting in the synthesis of bassanite (sample #2) and a mixture of bassanite and gypsum (sample #12), suggests a mechanism for producing bassanite under atmospheric conditions in tailings: that is, by the evaporation of calcium sulfate-rich water in a tailings pond from a warm solution (heated by a summer sun). If the bassanite does not come in contact with water after its formation, it will remain indefinitely.

In order to indicate whether or not the temperature of evaporation had an influence on the type of calcium sulfate obtained, a simple experiment was undertaken. Both sample #2 and #12 were leached with distilled water, and the leach solutions were allowed to evaporate to dryness at room temperature. White residues were obtained that consisted of clumps of well formed needles, which were identified as gypsum by x-ray diffraction.

It is therefore concluded that gypsum crystallizes from solution at a lower temperature than bassanite.

No well developed pisoliths, such as those developed in Key Lake tailings were identified in the Rabbit Lake samples.

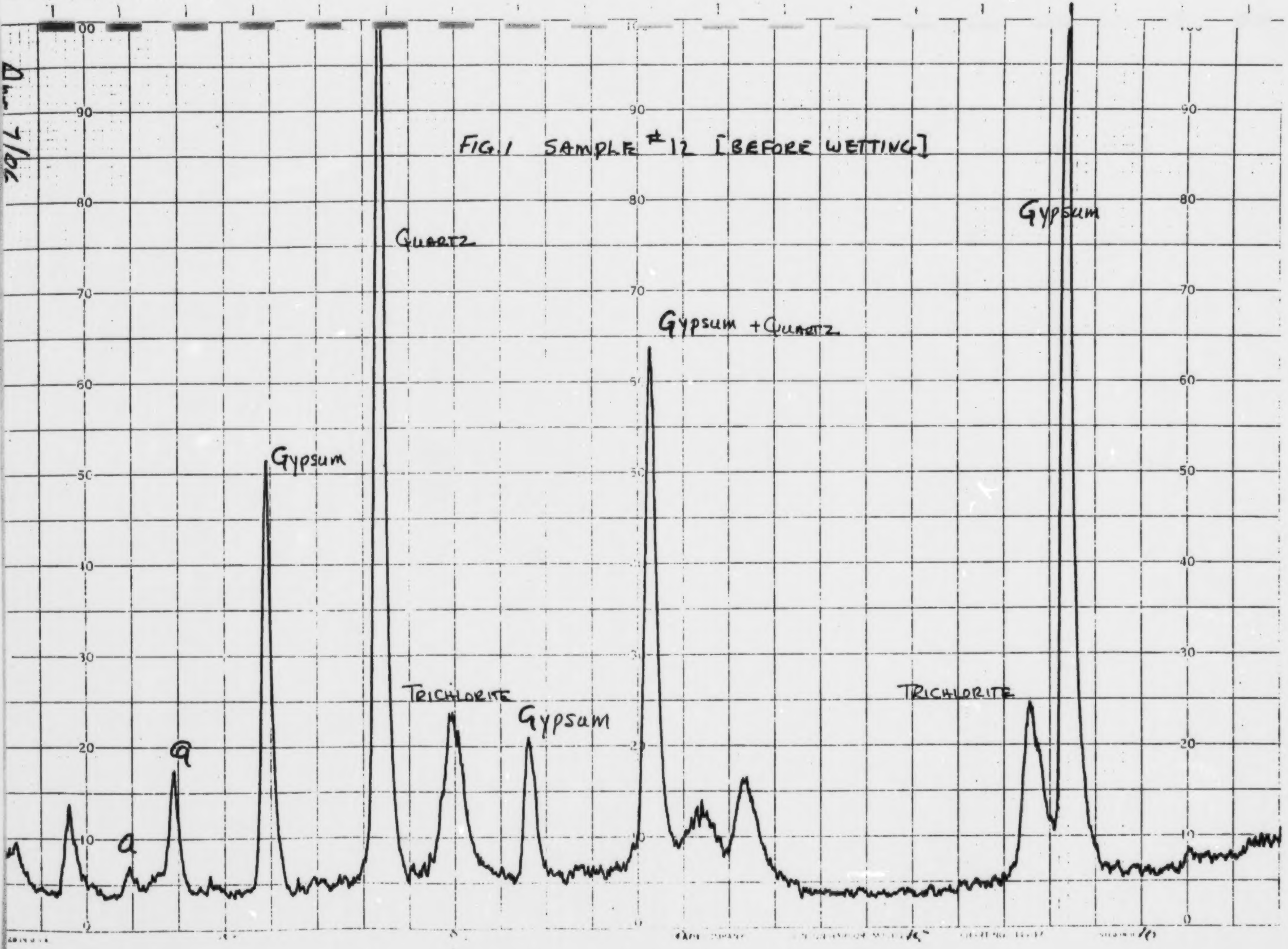


FIG. 1 SAMPLE # 12 [BEFORE WETTING]

Dun 7/06

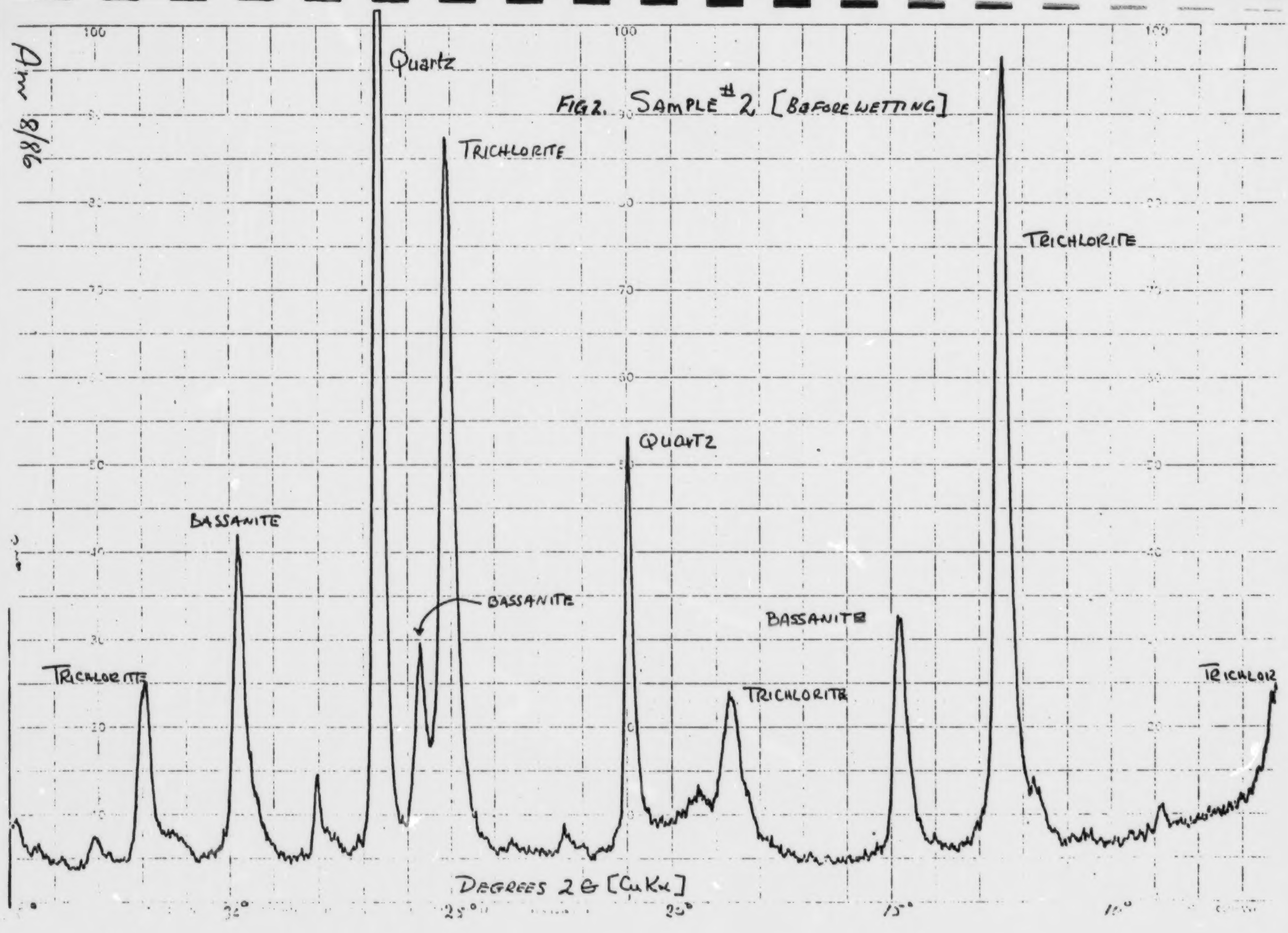


FIG 2. SAMPLE # 2 [BEFORE WETTING]

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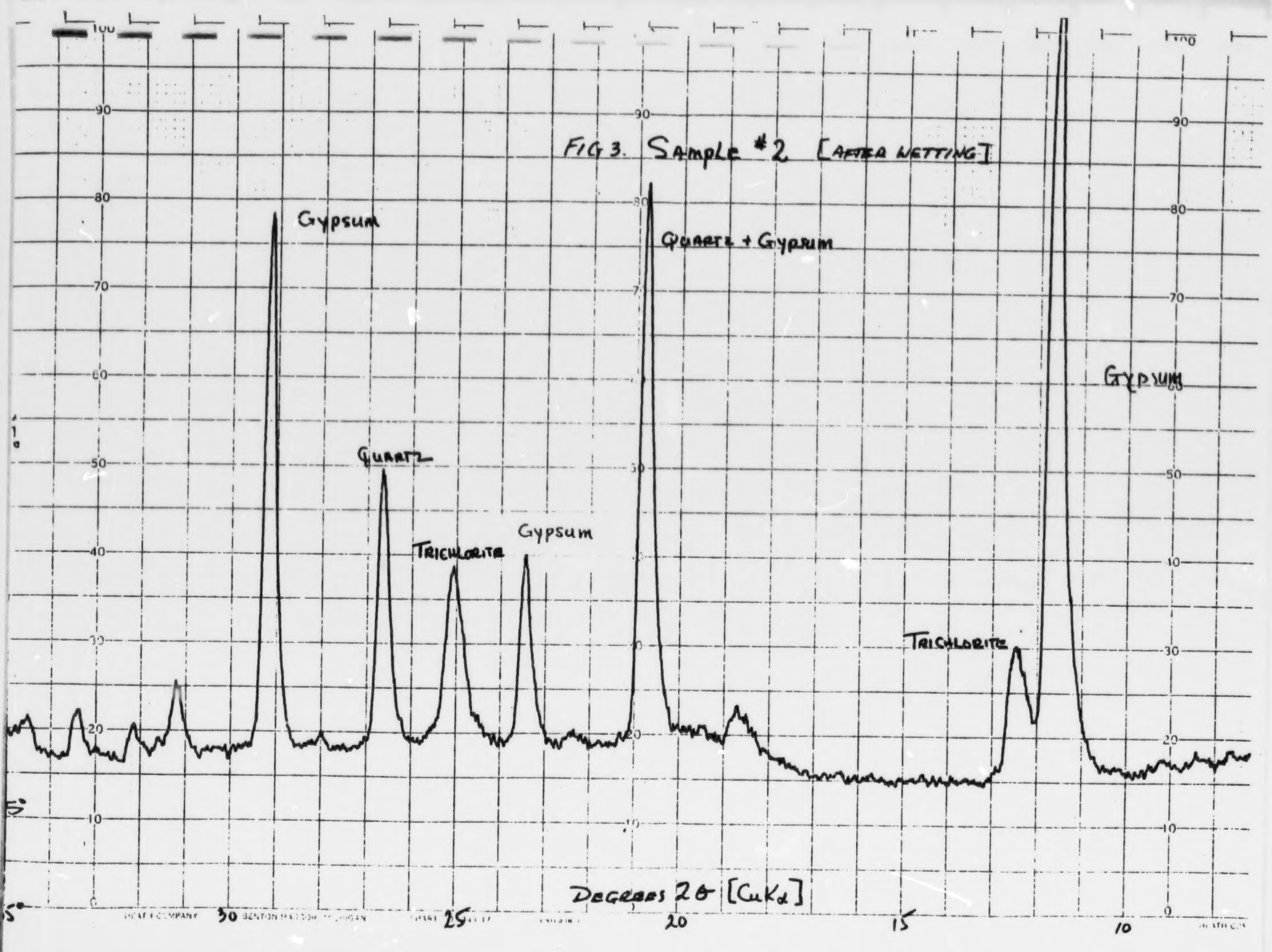
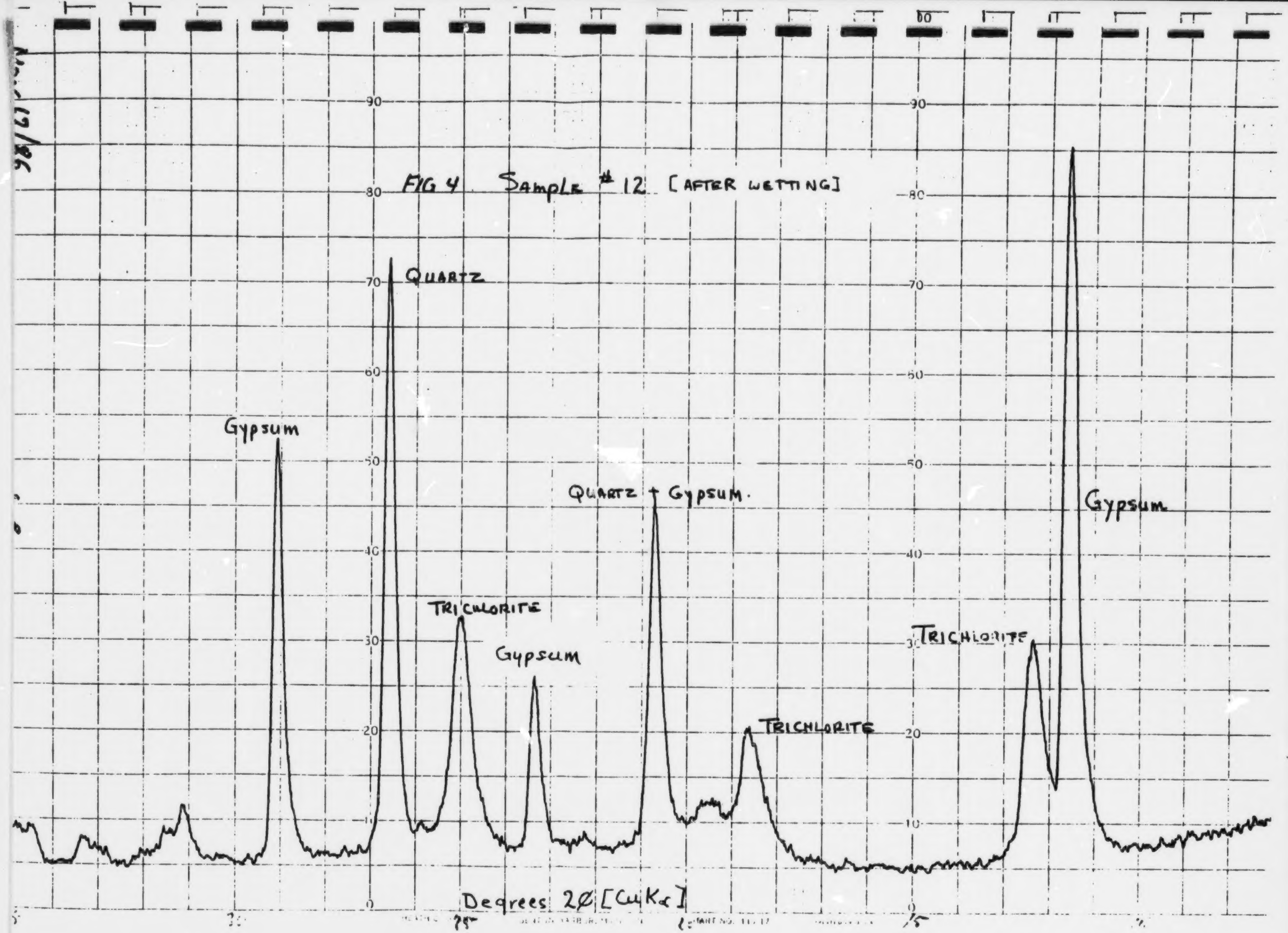


FIG 3. Sample #2 [After wetting]

 FIG 4 Sample # 12 [AFTER WETTING]

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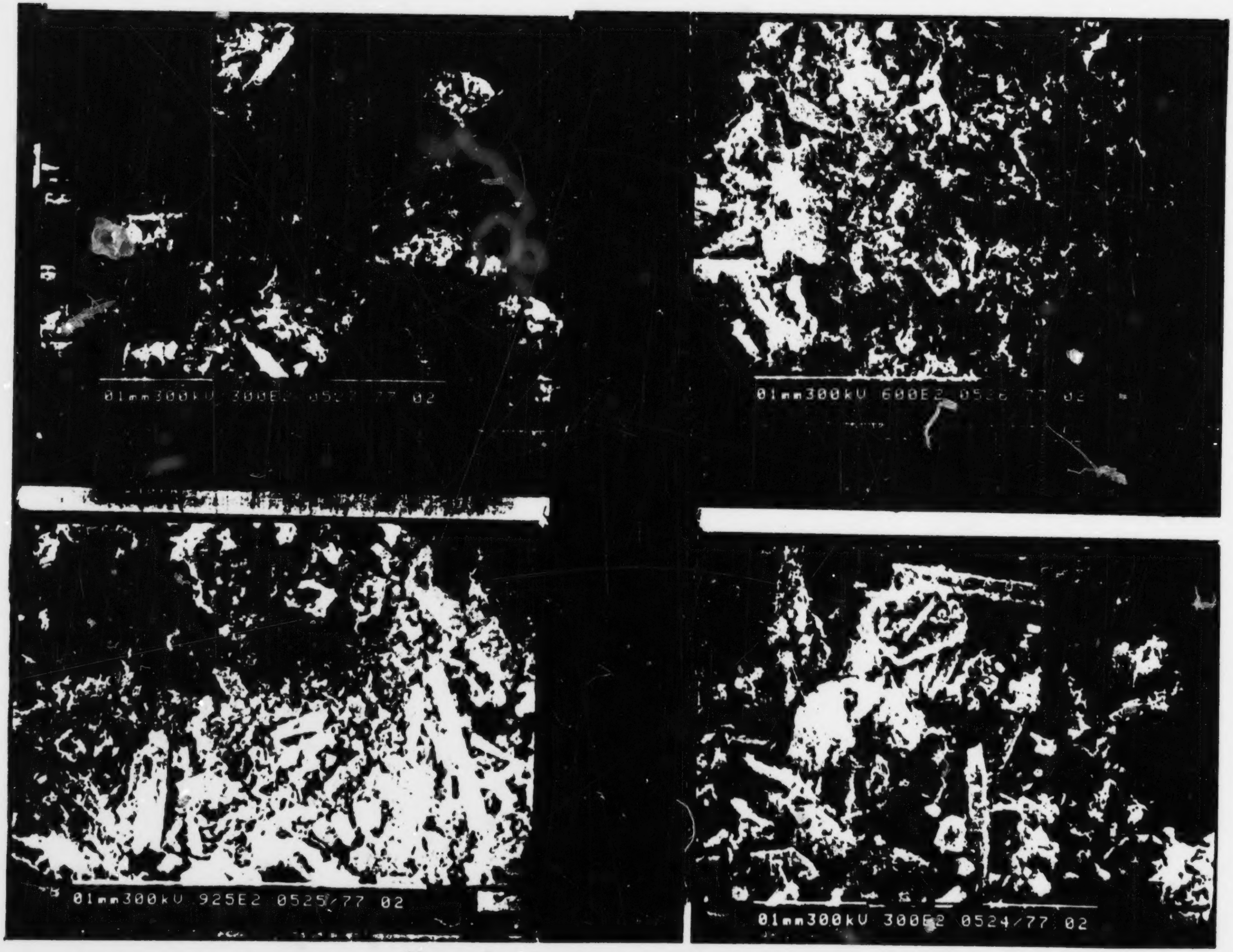
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01mm300kV 600E2 0526/77 02
01mm300kV 925E2 0525/77 02
01mm300kV 300E2 0524/77 02

Figure 5. Sem Photos of Sample 2.

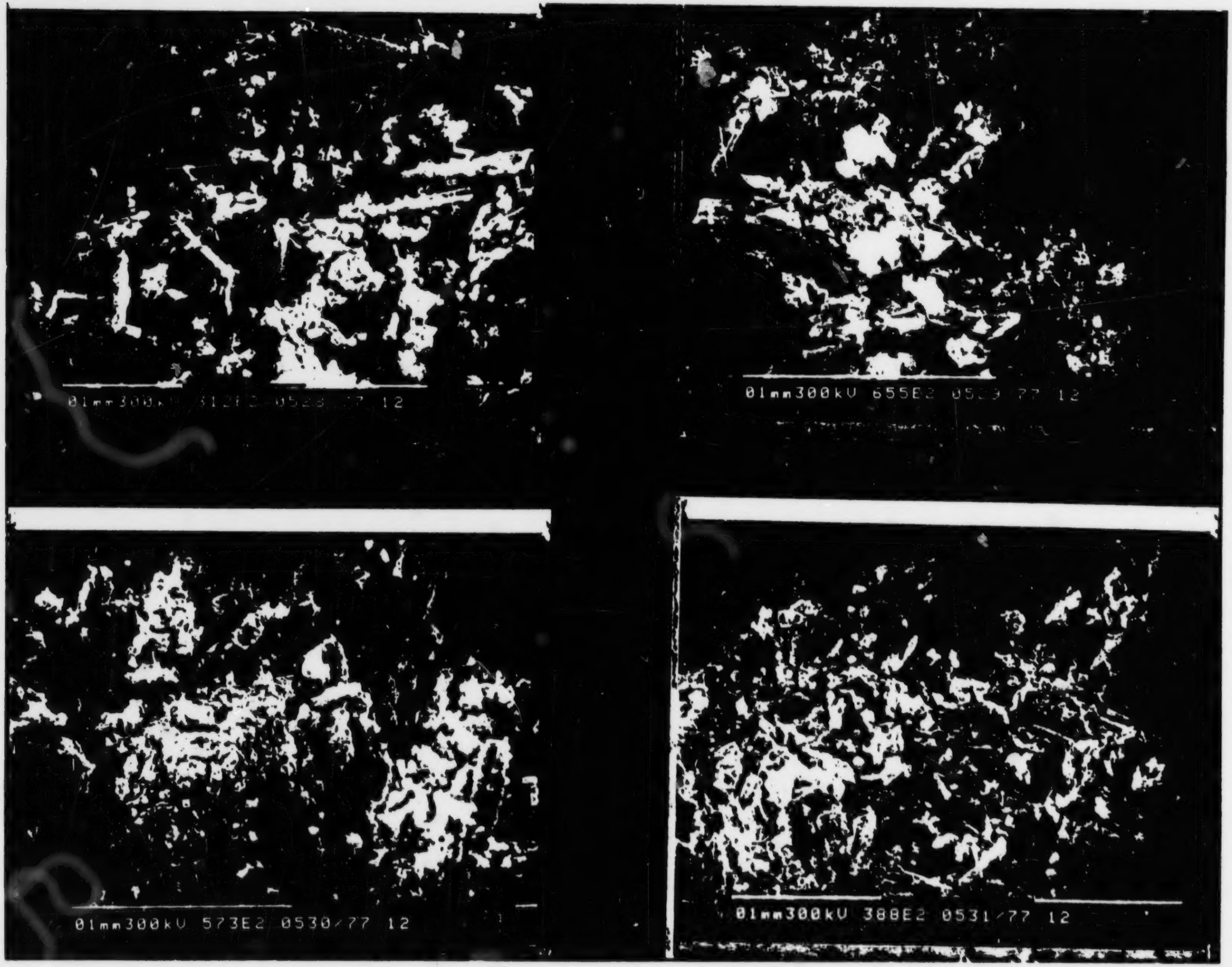
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Figure 6. Sem Photos of Sample 12.

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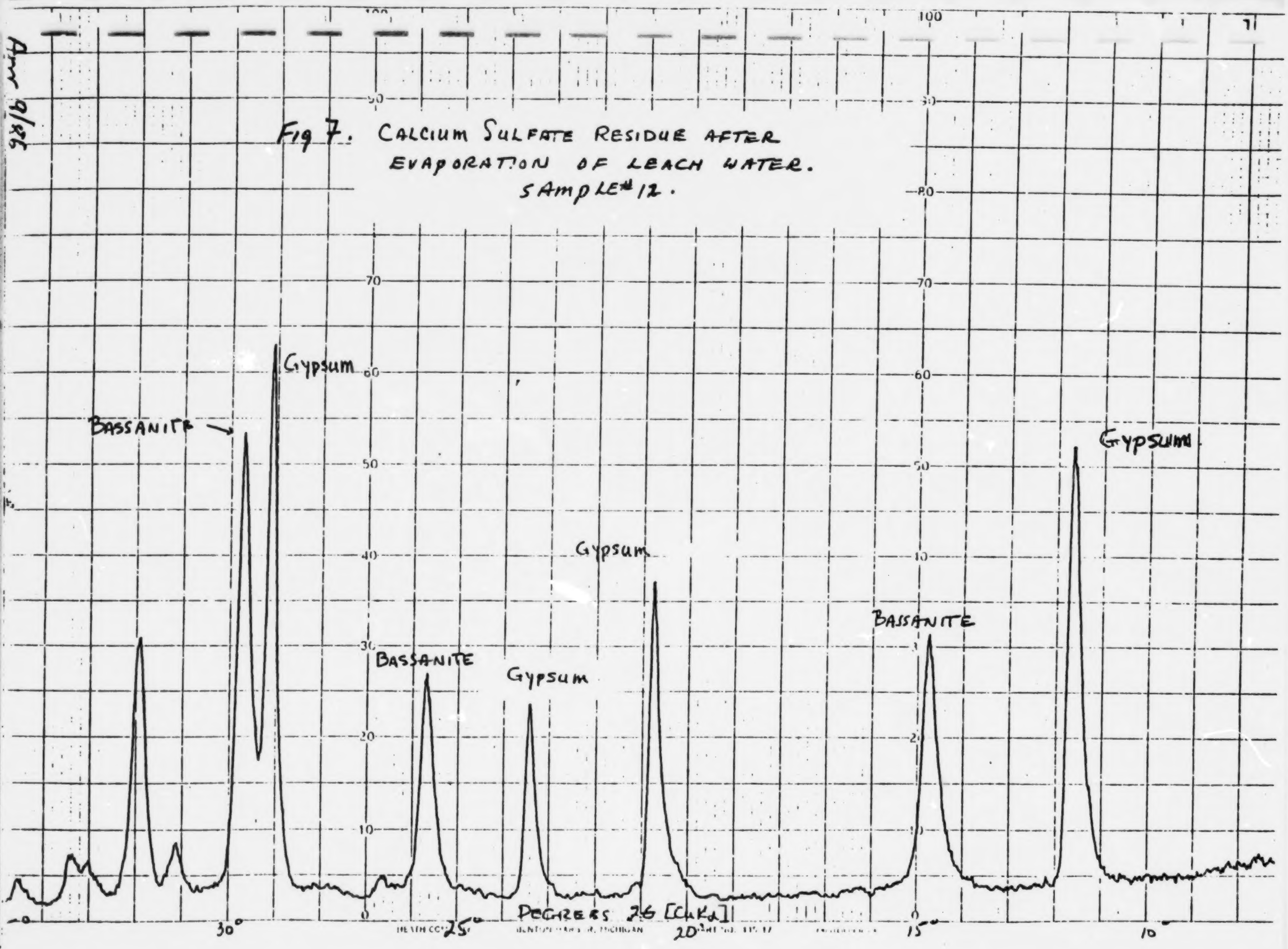

Fig 7. Calcium Sulfate Residue after evaporation of leach water. Sample# 12.

CHEMICAL ANALYSIS OF THREE RABBIT LAKE TAILINGS SAMPLES

BOREHOLE 2: SAMPLES 2, 8 AND 11

by

R.G. Arnold

SRC Publication No. R-855-7-C-86

September, 1986

INTRODUCTION

Three core samples of Rabbit Lake tailings were received for analysis in July, 1986. The samples were enclosed in metal pipes whose ends were covered with plastic. Only one pipe was plugged at one end with wax. The samples were identified as:

Borehole 2 Sample 2
Borehole 2 Sample 8
Borehole 2 Sample 11

For the sake of convenience, the samples are identified in this study as samples 2, 8 and 11.

The purpose of this study was (1) to document the composition and mineralogy of the samples as received, (2) determine the leaching of Ca, Mg, Al, Si, Ra^{226} and Th^{230} from the samples in slurry tests carried out for various time periods, (3) document the composition and mineralogy of grain size splits of the samples after leaching.

SAMPLE DESCRIPTION

Examination of the three samples (#2, #8, #11) microscopically showed that they were mineralogically similar. They consisted of a mixture of sub-angular to angular grains of quartz and other silicates, chlorite flakes, sulfides, hematite, dark colored metallics and abundant fine grained particles including clay minerals which coat the larger grains. Calcium sulfate, identified as bassanite by x-ray diffraction, occurred as small white translucent needles up to about 100u long and 15u wide. Scanning Electron Microscope photomicrographs of the three samples as received are shown in Fig. 1A, Fig. 2A and Fig. 3A.

The samples were prepared for analysis by splitting each core sample in half lengthwise and blending one-half to produce a homogeneous starting material for analysis.

Each blended sample was examined by X-ray diffraction using a dry mounting procedure so that the calcium sulfate mineral in the cores as received would not be altered. The X-ray diffraction charts shown in Figs. 4, 5 and 6 indicate that the main constituents of the samples are quartz and chlorite and a small quantity of bassanite. Fig. 7 shows an X-ray diffraction chart of a gypsum sample. Comparison of Fig. 7 with Figs. 4, 5, and 6 indicates the absence of gypsum in the three core samples.

LEACHING EXPERIMENTS

The leaching experiments were performed for 0.5, 4, 15 and 48 hours periods. De-ionized or distilled water was added to weighed samples to make a slurry which was continuously stirred for the leaching period. Distilled water (pH=4.95) was used for the 48 hour leach, and de-ionized water (pH=5.70) was used for the remaining leaching experiments. The proportion of solid sample to water was close to 5.2g/l for all leaching experiments. This proportion was used so that there would be less than 1 gram/litre of calcium sulfate (judged from the analysis of the original samples) which ensured that the concentration of calcium sulfate would be below the limit of saturation in solution. Only the liquid from the leaching experiments at 0.5, 4 and 15 hour periods was analyzed. Both the liquid and solid were analyzed from the leaching experiment conducted for the 48 hour period.

The solid residues from the 48 hour leaching experiments were separated into a sand size fraction (+44u)* by sieving, and into silt (44 - 5u) and clay (-5u) fractions by a sedimentation method. Scanning Electron Microscope pictures, shown in Figs. 1, 2 and 3 (photomicrographs B to D), were taken of each size fraction obtained from the leach residues.

RESULTS OF EXTRACTIONS

The chemical analyses of the original three solid samples and of the liquids and solids from the leaching experiments are summarized in Tables 1, 2 and 3. To aid in the interpretation of the results, the metal concentrations in the liquids are expressed as "% extracted" in the liquid relative to the concentration of the metal in the original solid sample. The percent metal extracted versus leach time for each metal is summarized on graphs in Figs. 8, 9 and 10. The data in the tables and graphs are easily interpreted in terms of the maximum concentrations extracted and the variation in concentration extracted with time. It will be left to the reader to extract from the data what he can, however a few comments are offered below.

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* These units are microns or micrometres.

Results for liquids:

The amount of Ca extracted was high and ranged from 84 to 100% of the amount available. Most of the Ca was dissolved within 0.5 hours indicating a highly soluble Ca mineral which must have been bassanite. The high rate of solution of bassanite was probably due to the small crystal size. The residual undissolved Ca in the solids, even after 48 hours leaching (sample #2 in particular), probably occurs in water insoluble silicates.

The extraction of Mg did not exceed 2.8% and was generally low. The Mg extracted appears to increase up to about 15 hours and then level off. The unextracted magnesium probably is contained in silicates (chlorite).

The amount of Al extracted was very low and all values were below 0.20 to 0.16% extracted. These values are related to the limits of detectability of Al under the experimental conditions; consequently all the graphs show zero slope as a function of leaching time. The unextracted aluminum is probably contained in clays and other silicate minerals.

The amount of Si extracted was very low with generally less than 0.14% of the available being extracted. There is a suggestion from the graphs that the extraction of Si may increase very slowly with time. The unextracted silicon is probably contained in quartz and silicate minerals.

The Ra^{226} results indicate that up to 33% of the available radium can be extracted within 0.5 hours and up to 35% can be extracted in 4 hours (sample #2). At longer times the percent extracted either stays constant with time (sample #11) or declines somewhat (samples #2 and #8).

The Th^{230} results indicate that only up to 2.8% of the available thorium was extracted within 0.5 hours (sample #2). The behavior of thorium extracted at 4 and 15 hours is somewhat erratic from one sample to another, but between 15 and 48 hours the percent extracted appears to increase for all samples.

Results for solids:

The grain size fractions were examined by the scanning electron microscope (Figs. 1, 2 and 3) and by X-ray diffraction (Figs. 11 to 19). The X-ray diffraction patterns show that the major minerals in the various fractions are quartz and chlorite with a trace of illite in some fractions. The other silicate and metallic minerals occur in concentration below the limit of detectability of the method (several percent of the sample).

The concentration of Ca in the solids is low (<0.42%) and decreases abruptly in the -5u fraction. This may be due to the removal of the last traces of calcium sulfate as a result of the method used in separating the various grain sizes. Mg and Al both increase and Si decreases as the grain size decreases. This may be due to the increase in the ratio of chlorite to silica with decreasing grain size.

Ra^{226} and Th^{230} both increase with decreasing grain size in all three samples. Whether or not this is due to an increase in concentration of a radium and thorium compound or due to surface adsorption on other minerals as the particle size decreases is not known.

CONCLUDING COMMENTS

Under the conditions of these experiments, that is, continuous agitation of slurries with a fixed solid to liquid ratio, the concentration of the various metals is solution attained their maximum or near maximum concentrations within 0.5 hours. Whether or not these are equilibrium concentrations under the leach conditions is not known. Certainly the concentration of silicon obtained exceeds the solubility of quartz in water at room temperature. Also, the chemistry of the radium and thorium containing compounds (if any) are unknown, and therefore a discussion of equilibrium concentrations is not possible.

The trends in the Ra^{226} and Th^{230} graphs (Figs. 8, 9 and 10), particularly from 15 to 48 hours are open to question. The thorium curves have a positive slope, whereas, two radium curves have a negative slope and the third remains essentially unchanged. In the case of thorium, positive slopes suggest increasing extraction with time. However, the rather erratic behavior of the concentration values between 0.5 and 15 hours begs the question as to whether or not the results are a reflection of methodology and or analysis. The average of all the values together probably have some significance. In the case of radium, the negative slopes could suggest the formation and precipitation of a secondary compound with increasing time. But again, it is suspected that this may be an artifact of the methodology and analysis, and all the values taken together probably have some significance.

Table 1 : Analytical results for sample # 2. Analytical results for sand (+44u), silt (44 - 5u) and clay (-5u) size fractions are after leaching.

	Ca	Mg	Al	Si	SO_4	Ra	Th
	%	%	%	%	%	*	*
original solid	1.56	7.4	5.1	29.0	3.46	10	11
0.5 hours							
liquid	1.32	.03	.01	.003	-	3.3	.31
% extracted	84.6	.41	.20	.01		33	2.8
4 hours							
liquid	1.32	.03	.01	.014	-	3.5	.10
% extracted	84.6	.41	.20	.05		35	.91
15 hours							
liquid	1.42	.04	.01	.026	-	3.0	.12
% extracted	91.0	.54	.20	.09		30	1.09
48 hours							
liquid	1.37	.04	.01	.033		1.2	.21
% extracted	87.8	.54	.20	.11		12	1.91
+44	.32	7.2	5.1	30.3		5	7
44 - 5u	.38	7.1	5.3	29.2		14	16
-5u	.08	9.2	9.3	17.9		85	95

* Ra^{226} and Th^{230} values are given in Bq/litre for liquids and Bq/gram for solids.

Table 2 : Analytical results for sample # 8. Analytical results for sand (+44u), silt (44 - 5u) and clay (-5u) size fractions are after leaching.

	Ca %	Mg %	Al %	Si %	SO_4 %	Ra *	Th *
original solid	3.55	7.5	6.0	24.5	8.77	55	55
0.5 hours							
liquid	3.41	.12	.01	.005	-	8.0	.16
% extracted	96.0	1.6	.17	.02		14.5	.29
4 hours							
liquid	3.40	.13	.01	.021	-	8.0	.14
% extracted	96.0	1.73	.17	.09		14.5	.25
15 hours							
liquid	3.45	.15	.01	.032	-	7.5	.10
% extracted	97.0	2.0	.17	.13		13.6	.18
48 hours							
liquid	3.35	.15	.01	.030		4.5	.49
% extracted	94.0	2.0	.17	.12		8.2	.89
+44u	.25	7.4	5.7	29.1		14	15
44 - 5u	.42	8.4	7.1	24.1		65	90
-5u	.05	7.6	8.6	15.8		140	200

* Ra^{226} and Th^{230} values are given in Bq/litre for liquids and Bq/gram for solids.

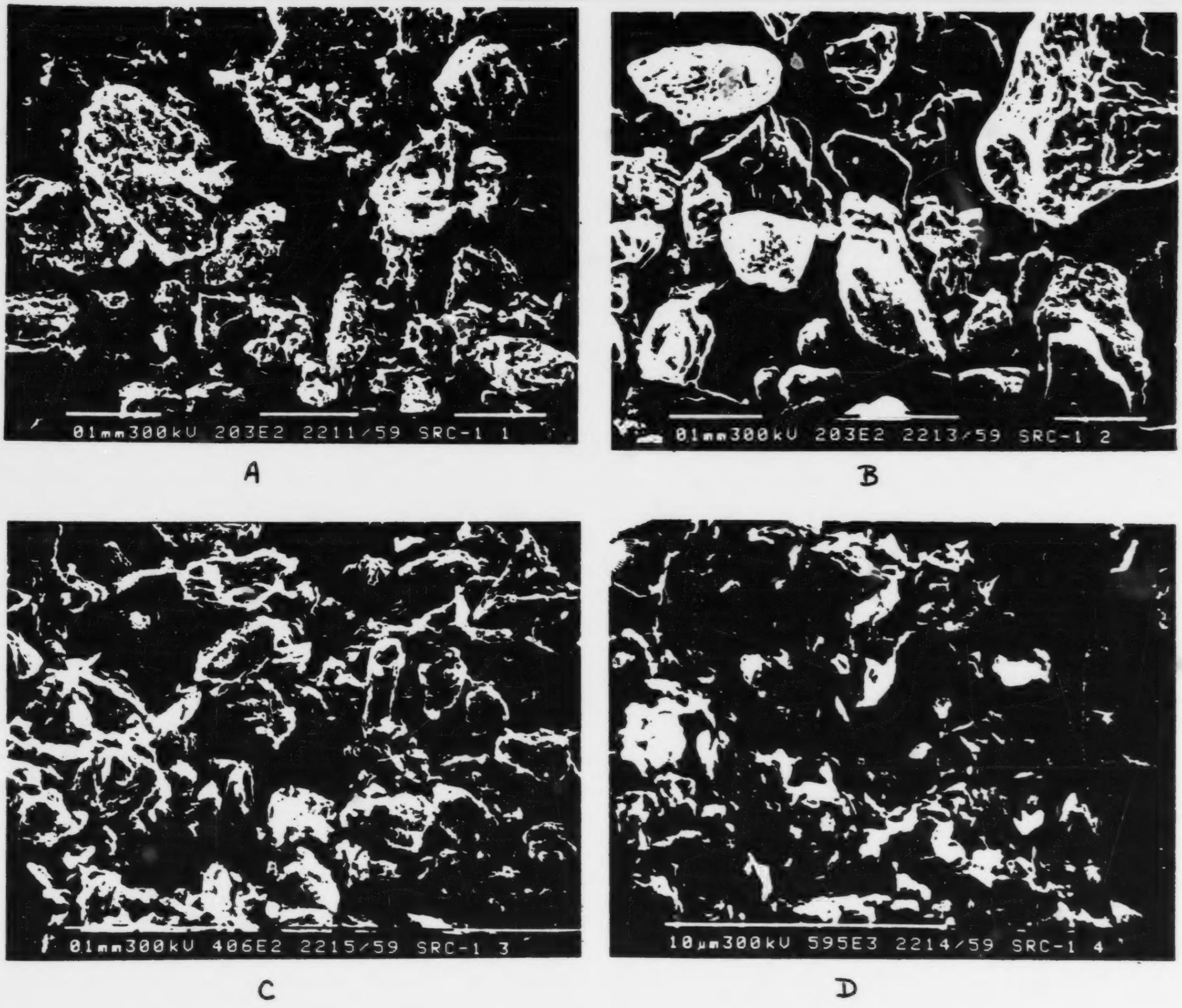
Table 3 : Analytical results for sample # 11. Analytical results for sand (+44u), silt (44 - 5u) and clay (-5u) size fractions are after leaching.

	Ca %	Mg %	Al %	Si %	SO_4 %	Ra *	Th *
original solid	4.90	7.5	6.2	19.5	13.0	55	60
0.5 hours							
liquid	4.80	.17	.01	.008	-	4.8	.08
% extracted	98.0	2.3	.16	.04		8.7	.13
4 hours							
liquid	4.80	.20	.01	.022	-	5.5	.10
% extracted	98.0	2.67	.16	.10		10.0	.17
15 hours							
liquid	4.90	.20	.01	.027	-	4.5	.08
% extracted	100	2.67	.16	.10		8.0	.13
48 hours							
liquid	4.80	.21	.01	.028	-	4.5	.56
% extracted	98.0	2.8	.16	.14		8.2	.93
+44u	.29	7.1	5.7	27.8		30	50
44 - 5u	.32	8.4	8.0	22.9		65	65
-5u	.07	8.8	10.0	16.1		100	140

* Ra^{226} and Th^{230} values are given in Bq/litre for liquids and Bq/gram for solids.

Table 4. Grain size separations of solids after leaching for 48 hours. The numbers indicate the percentage of the material in each grain size fraction.

	Grain Size Fraction			
Sample	+44u	44 - 5u	-5u	
#2	78.7	17.5	3.8	= 100%
#8	32.1	64.2	3.7	= 100%
#11	18.7	67.0	14.3	= 100%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A
01mm300kV 203E2 2213/59 SRC-1 2
B
01mm300kV 406E2 2215/59 SRC-1 3
C
10µm300kV 595E3 2214/59 SRC-1 4
D

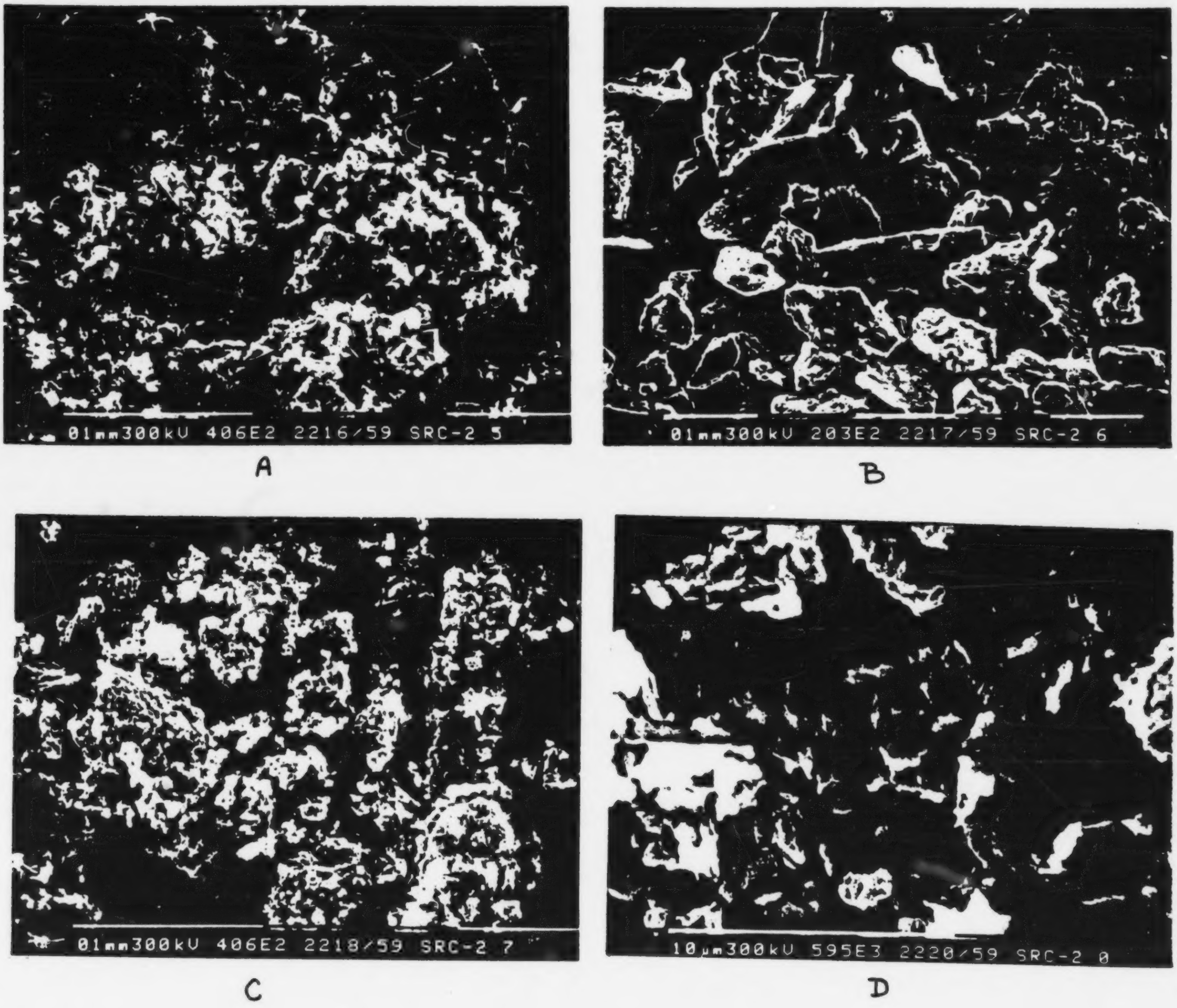
Fig. 1. SEM photomicrographs of sample #2.

A. Sample as received. Large quartz and silicate grains coated with fine grained minerals and clays. A few bassanite needles are visible. (203X magnification).

B. +44u fraction after leaching for 48 hours. Large quartz and silicate grains and a few chlorite flakes. (203X)

C. 44 - 5u fraction after leaching for 48 hours. Quartz and silicate grains coated with fine-grained minerals and clays. (406X)

D. -5u fraction after leaching for 48 hours. Fine grained minerals and clays. (5950X)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A
01mm300kV 203E2 2217/59 SRC-2 6
B
01mm300kV 406E2 2218/59 SRC-2 7
C
10μm300kV 595E3 2220/59 SRC-2 0
D

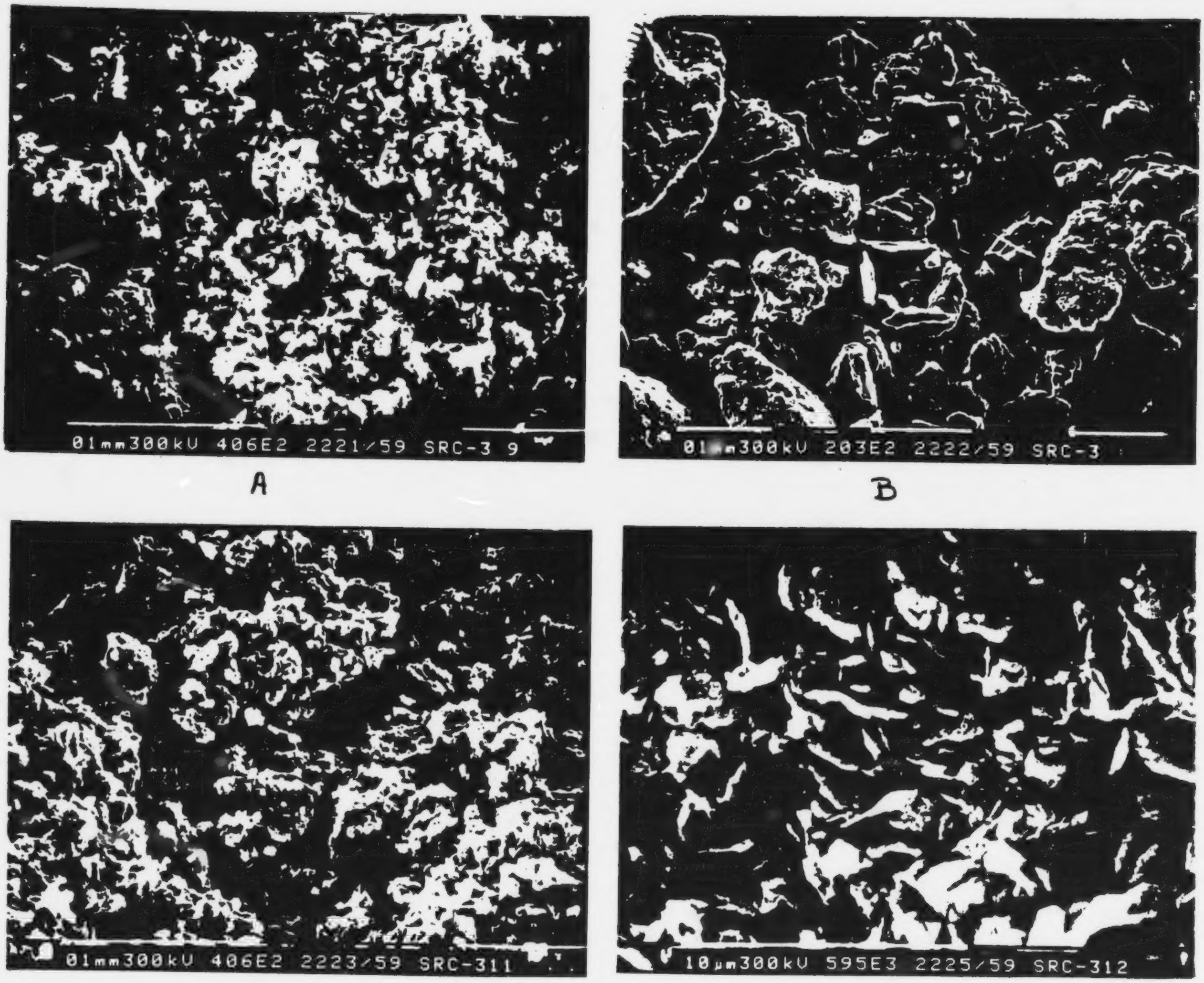
Fig. 2. SEM photomicrographs of sample #8.

A. Sample as received. Quartz and silicate grains coated with fine-grained minerals and clays. A few bassanite needles are visible. (406X).

B. +44u fraction after leaching for 48 hours. Quartz and silicate grains with a few chlorite flakes. (203X).

C. 44 - 5u fraction after leaching for 48 hours. Quartz and silicate grains coated with fine-grained minerals and clays. (406X).

D. -5u fraction after leaching for 48 hours.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01 m300kV 203E2 2222/59 SRC-3
01mm300kV 406E2 2223/59 SRC-311
10µm300kV 595E3 2225/59 SRC-312

A

B

C

D

Fig. 3. SEM photomicrographs of sample #11.

A. Sample as received. Quartz and silicate grains coated with fine-grained minerals and clays. A few bassanite needles are visible. (406X).

B. +44u fraction after leaching for 48 hours. Quartz, silicates and a few chlorite flakes. (203X).

C. 44 - 5u fraction after leaching for 48 hours. Quartz and silicates coated with fine-grained minerals and clays. (406X).

D. -5u fraction after leaching for 48 hours. Fine grained minerals and clays. (5950X).

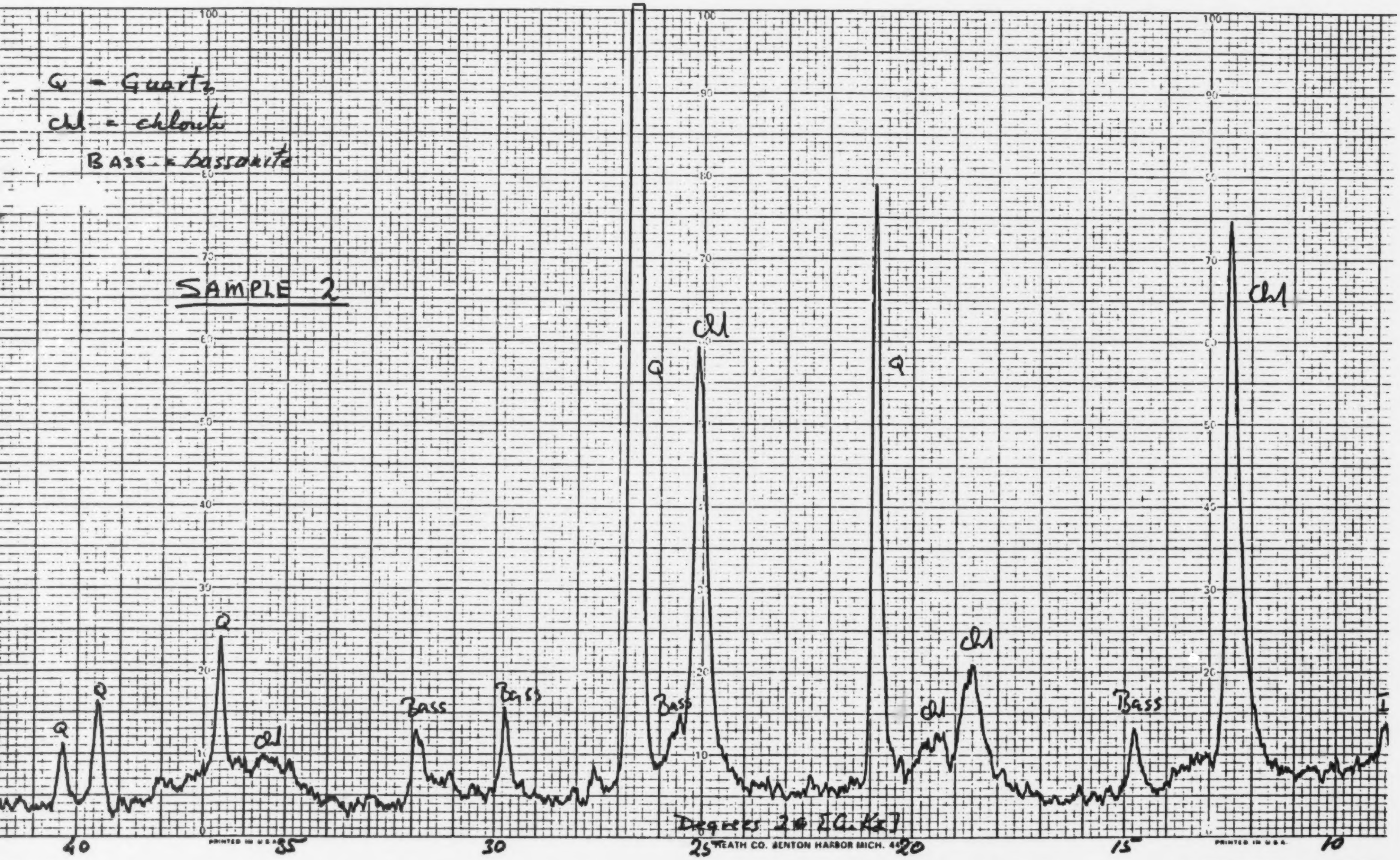

Fig. 4 XRD chart of sample #2 as received, showing abundant quartz, chlorite and some bassanite.

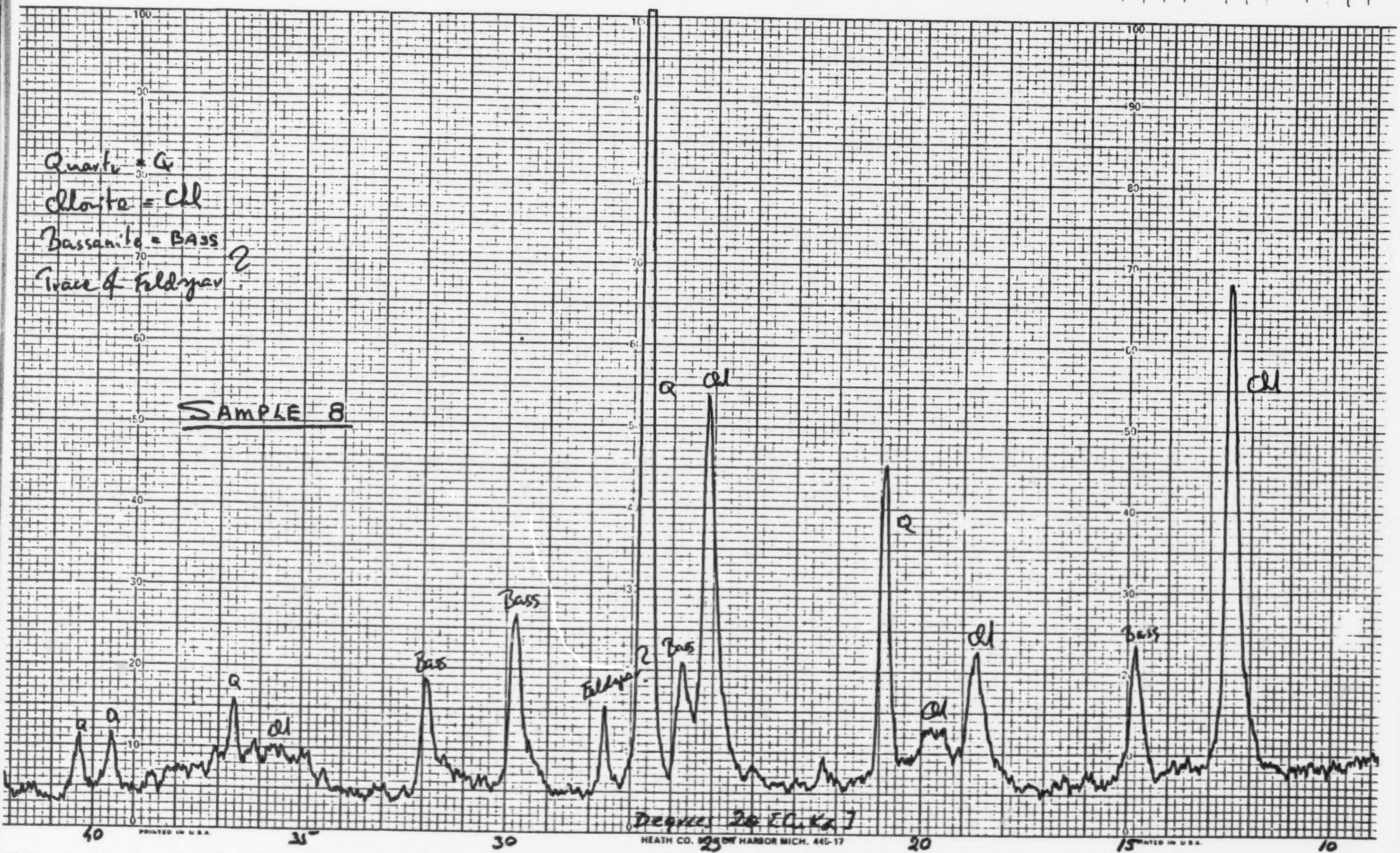

Fig. 5 XRD chart of sample #8 as received, showing abundant quartz, chlorite and some bassanite.

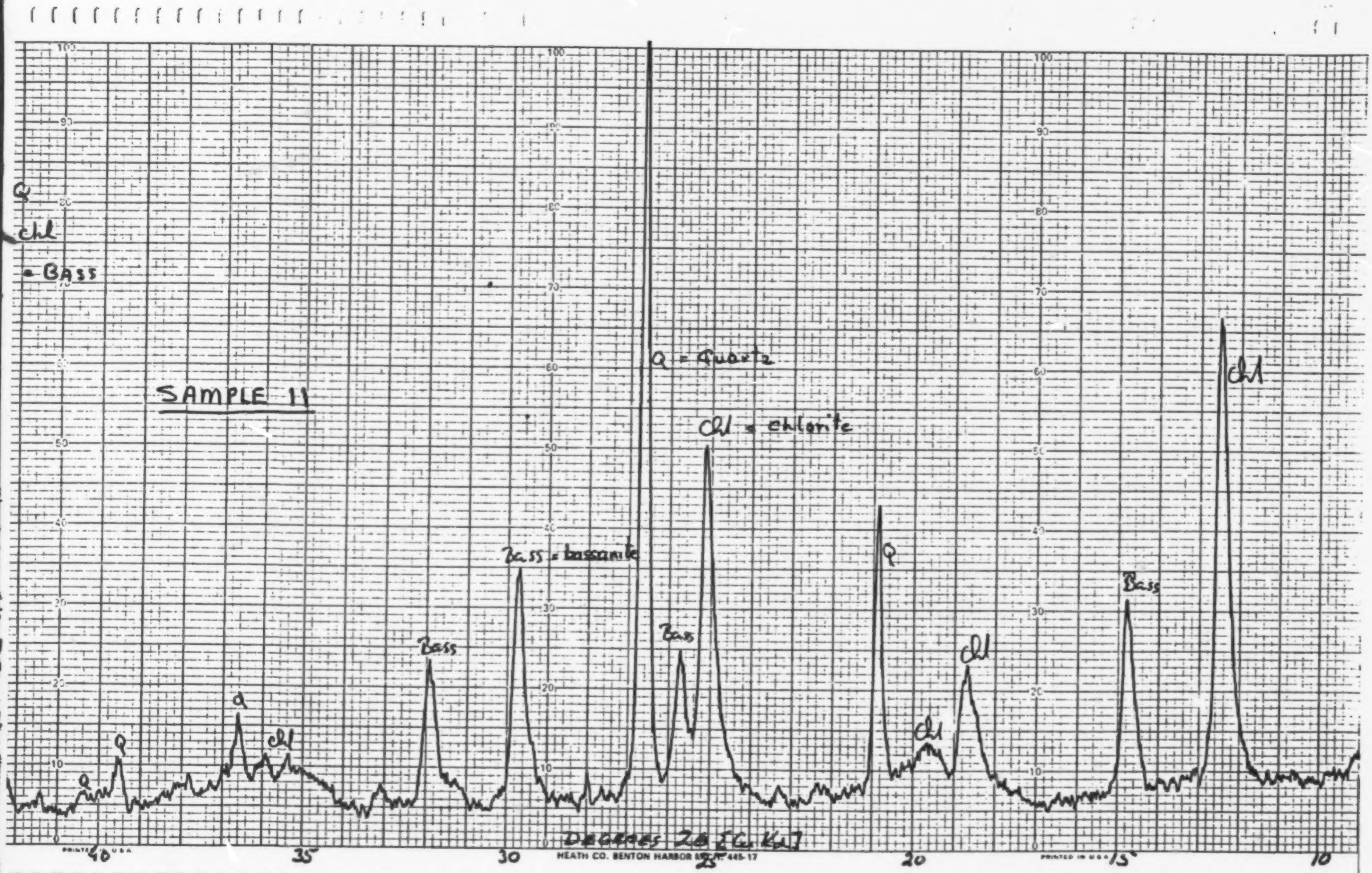

Fig. 6 XRD chart of sample #11 as received, showing abundant quartz, chlorite and some bassanite.

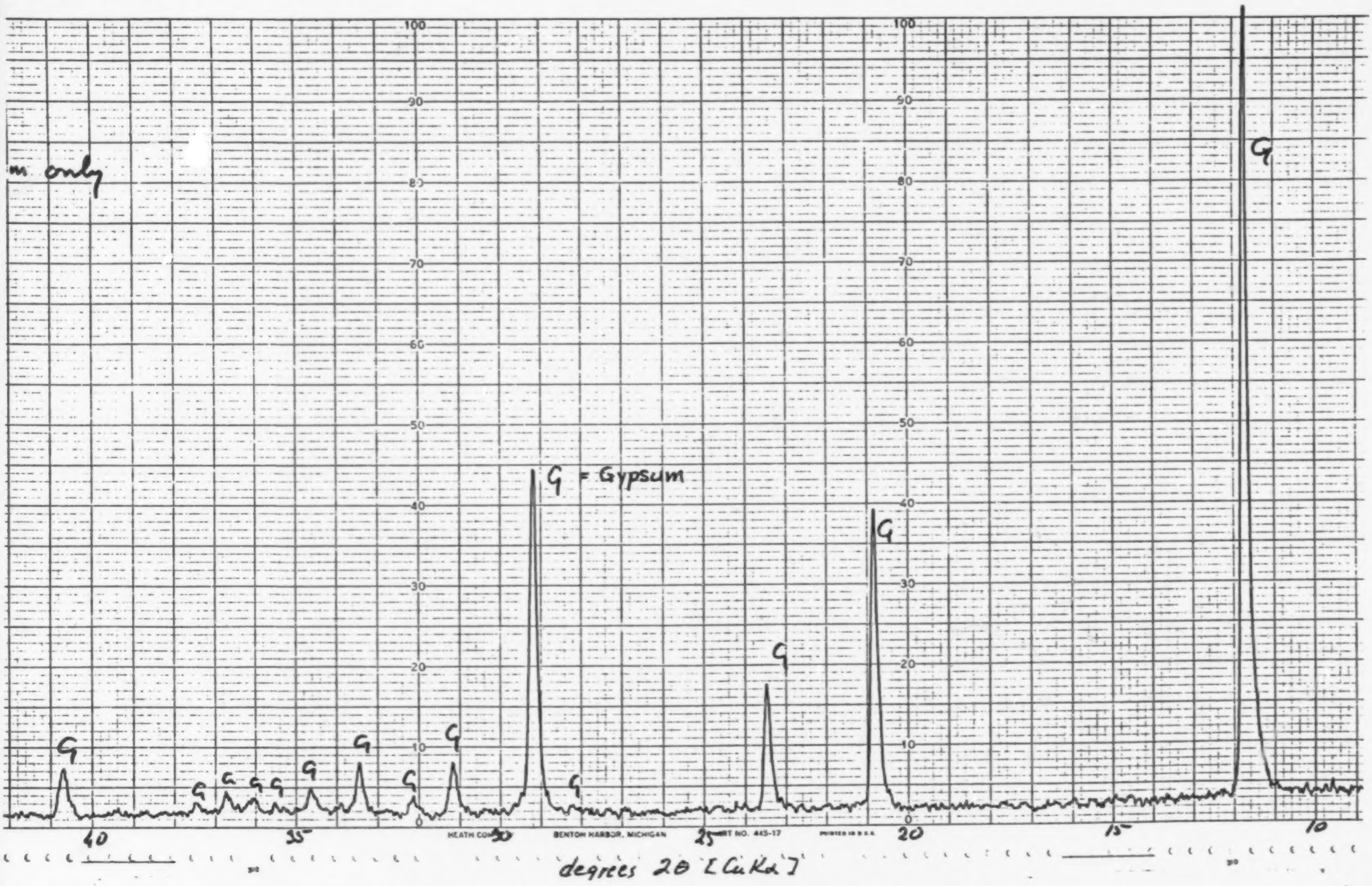

Fig. 7 XRD chart of a gypsum sample.

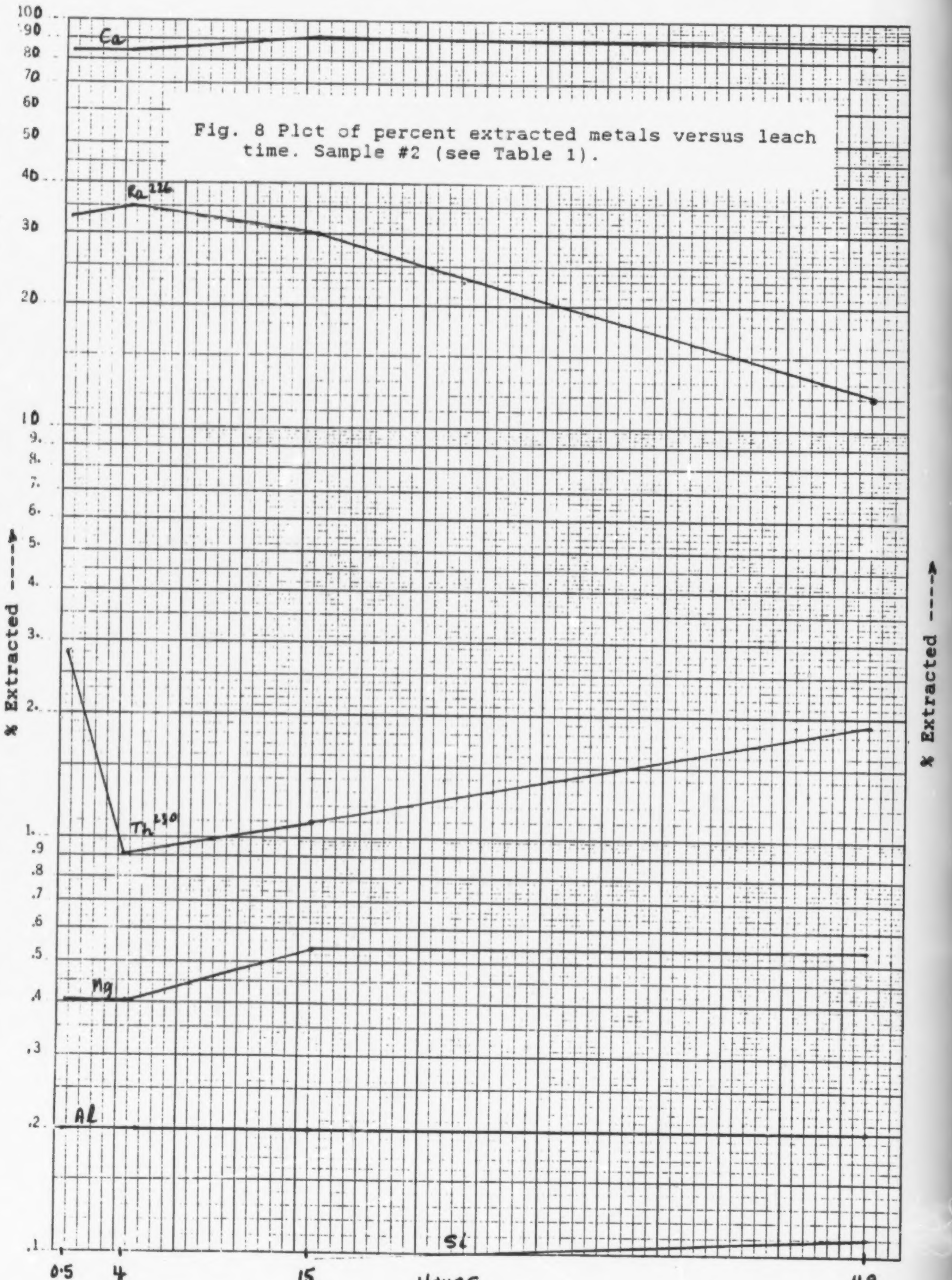

Fig. 8 Plct of percent extracted metals versus leach time. Sample #2 (see Table 1).

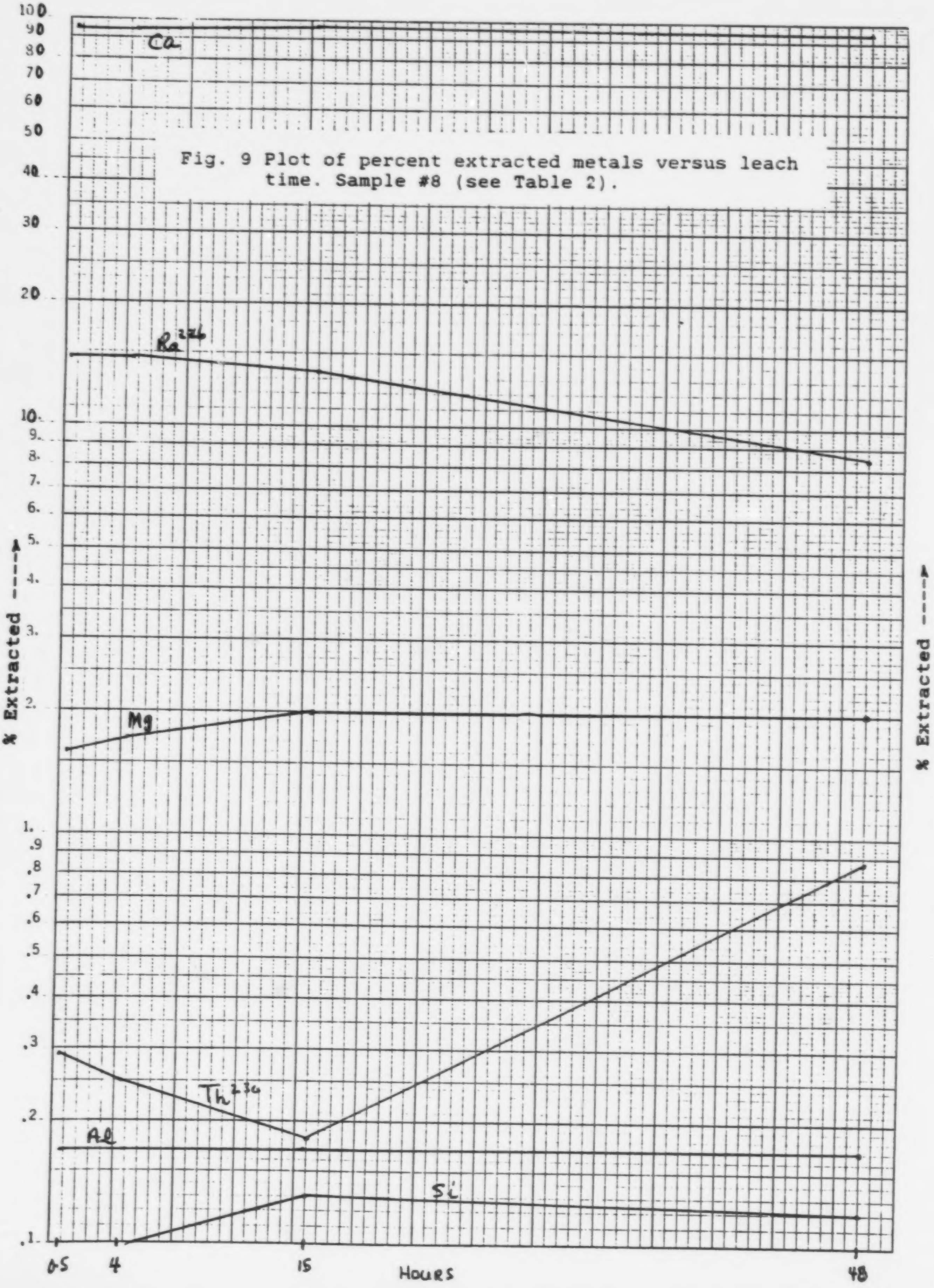
Fig. 9 Plot of percent extracted metals versus leach time. Sample #8 (see Table 2).

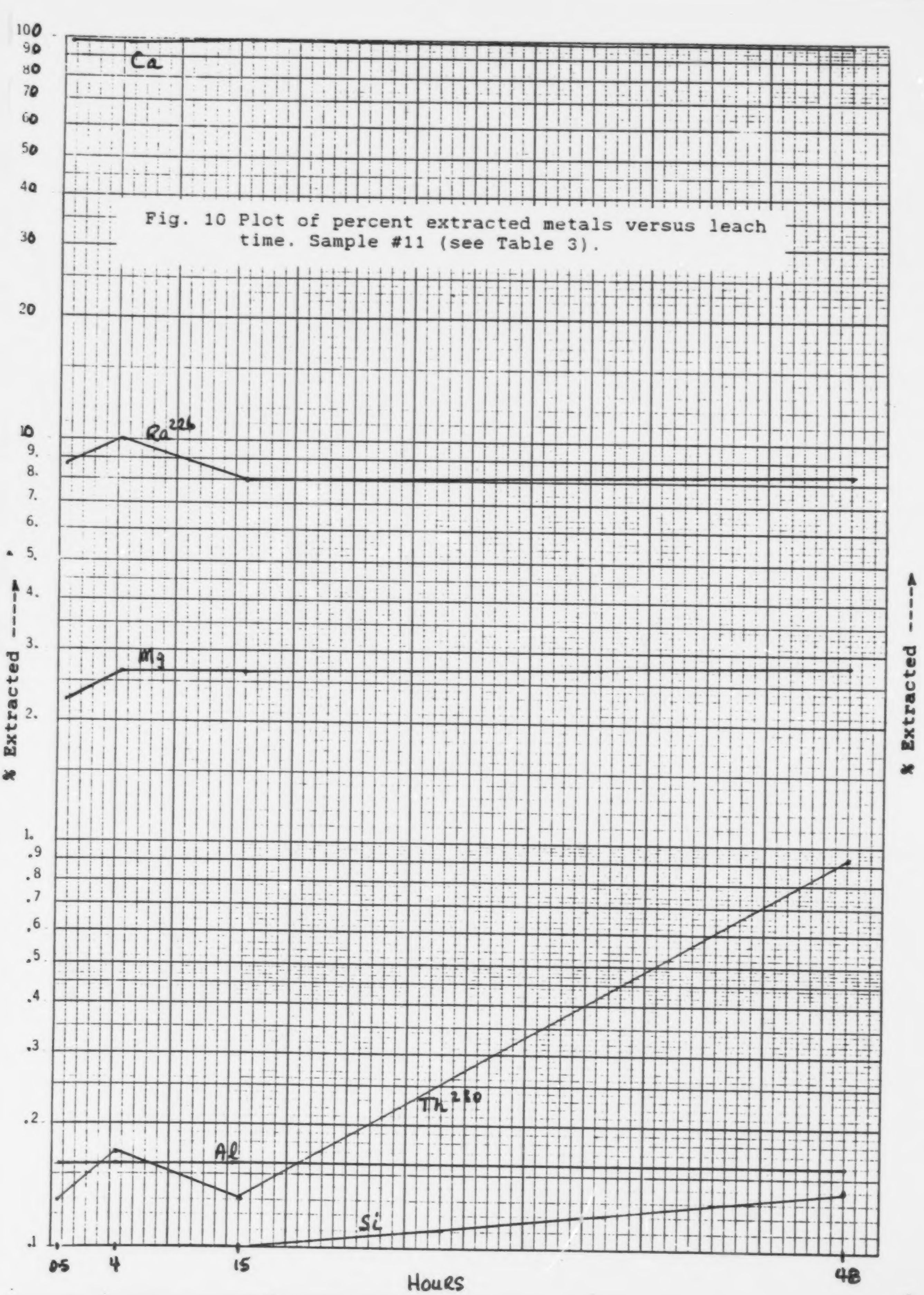

Fig. 10 Plot of percent extracted metals versus leach time. Sample #11 (see Table 3).

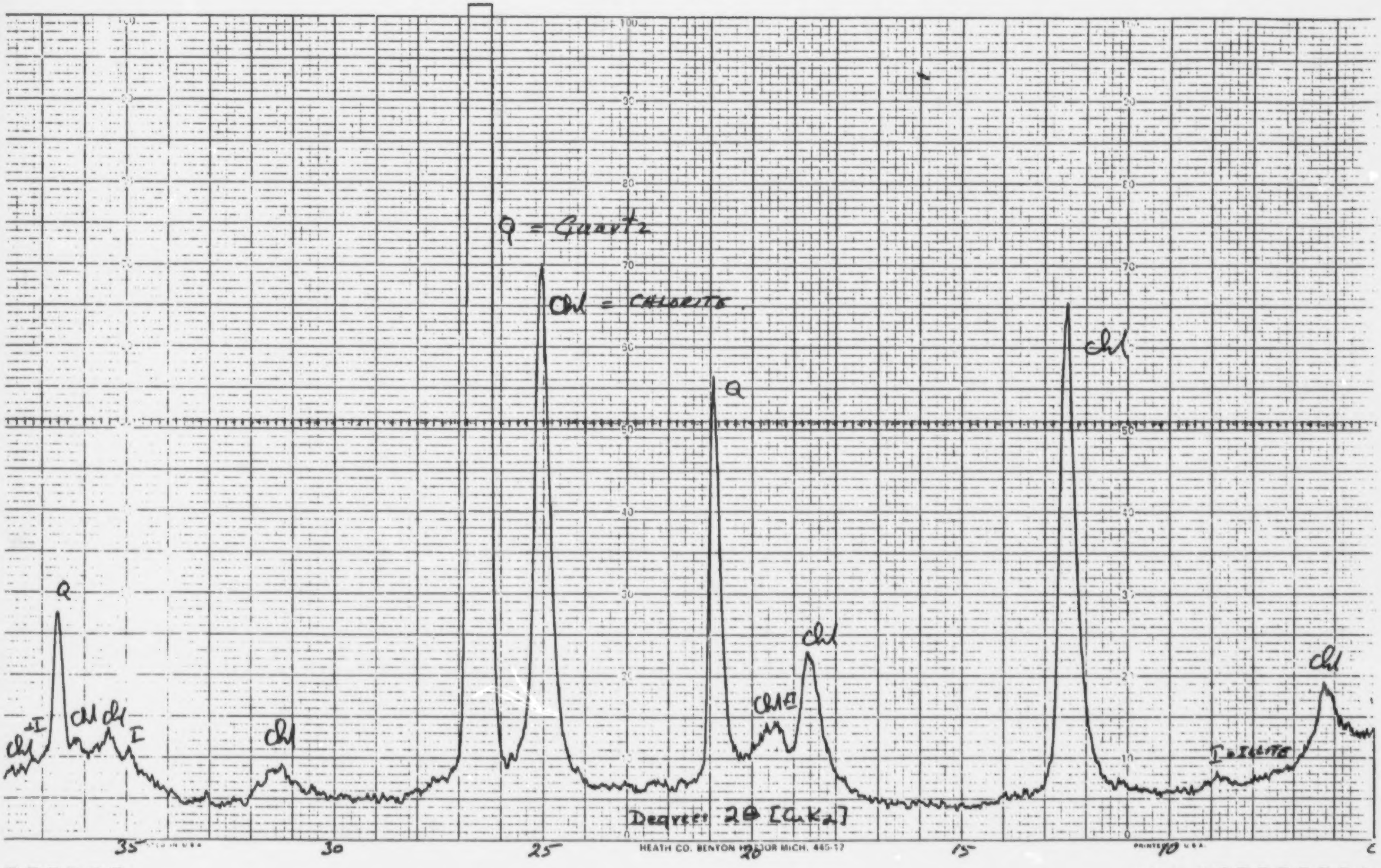

Fig. 11 XRD chart of the +44u fraction of sample #2 after 48 hour leach. Chart shows abundant quartz, some chlorite and a trace of illite.

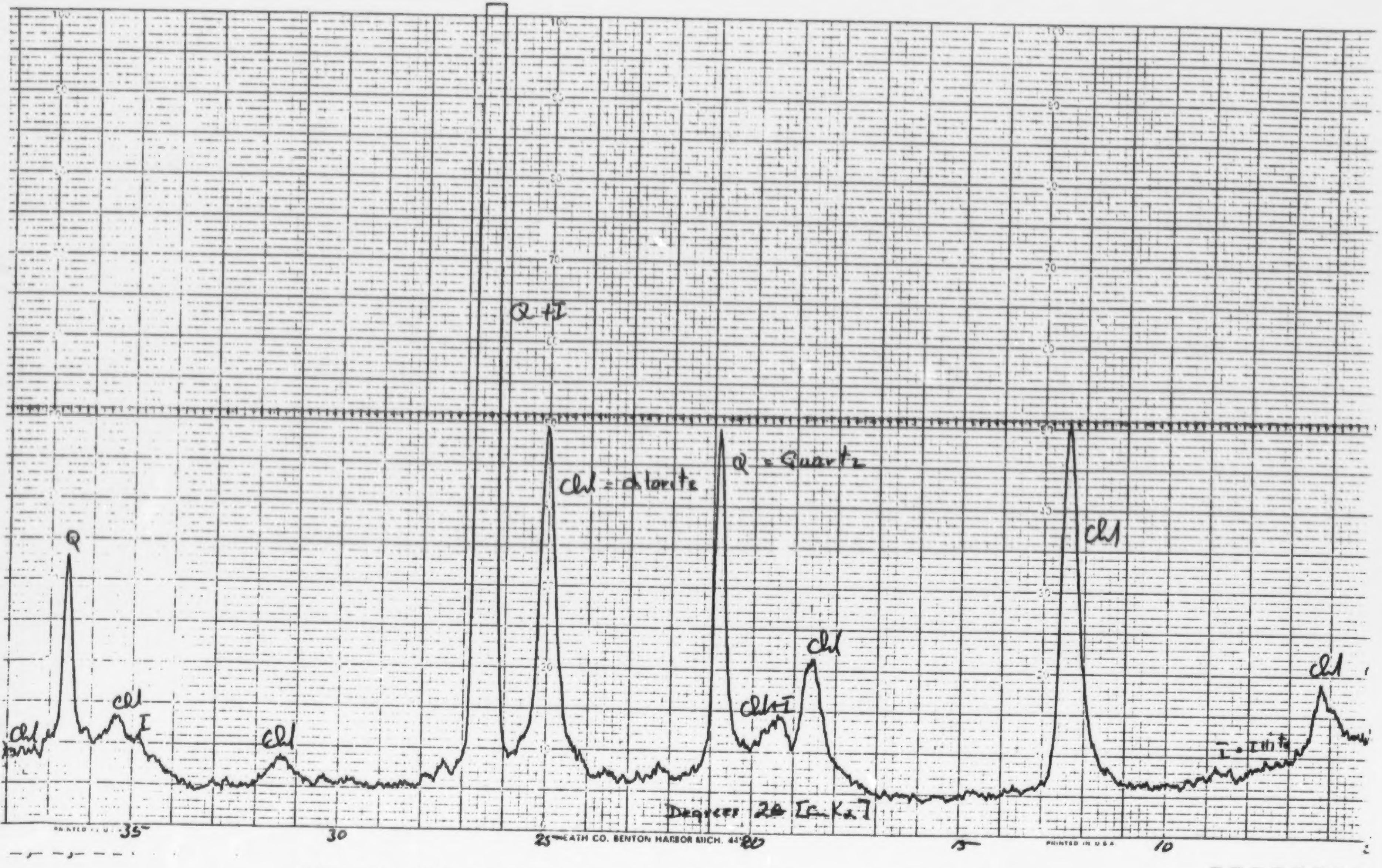

Fig. 12 XRD chart of the 44 - 5u fraction of sample #2 after 48 hour leach. Chart shows abundant quartz, some chlorite and a trace of illite.

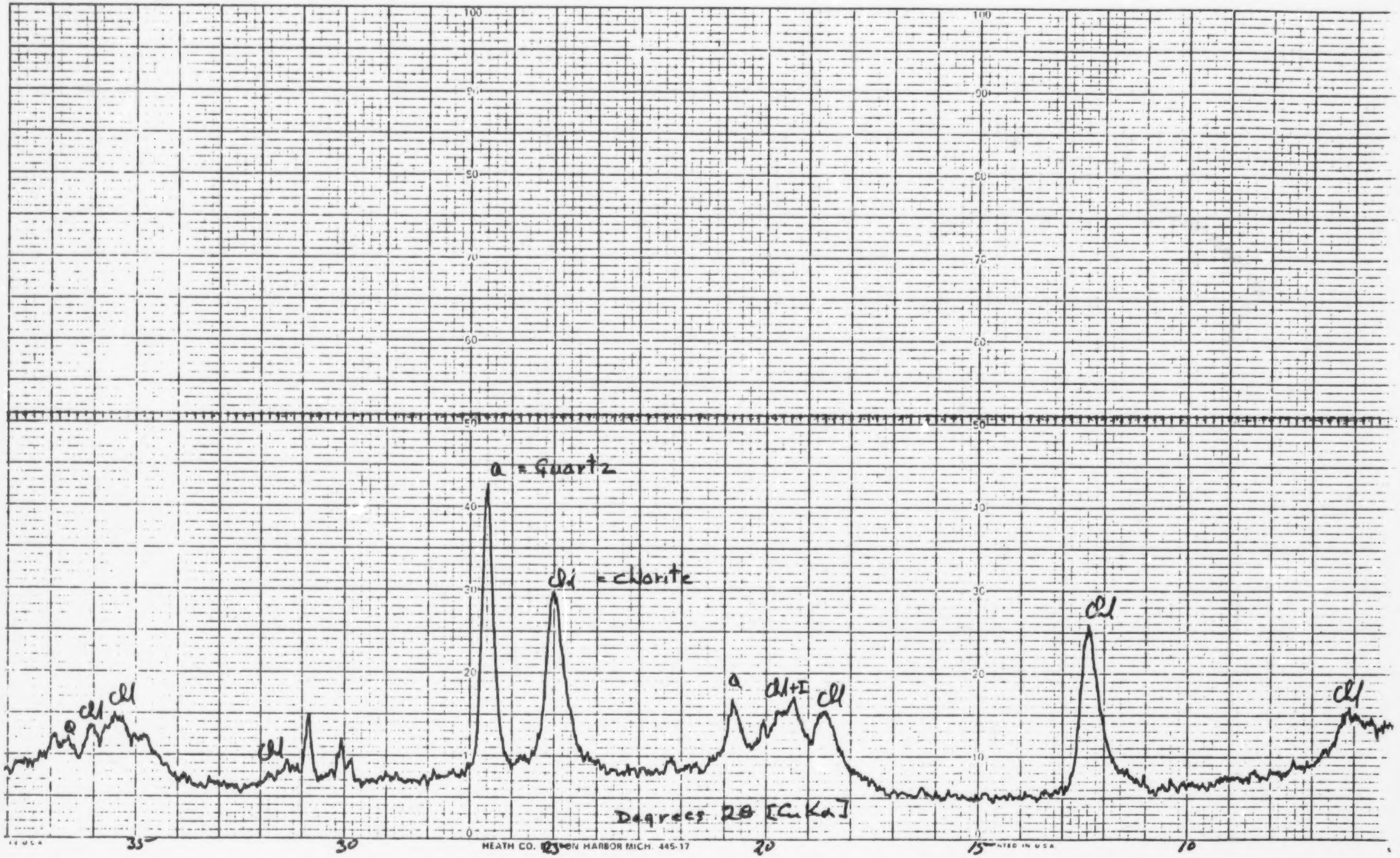

Fig. 13 XRD chart of the -5u fraction of sample #2 after 48 hour leach. Chart shows abundant quartz and chlorite. Reduced peak intensities are probably due to the fine grain size

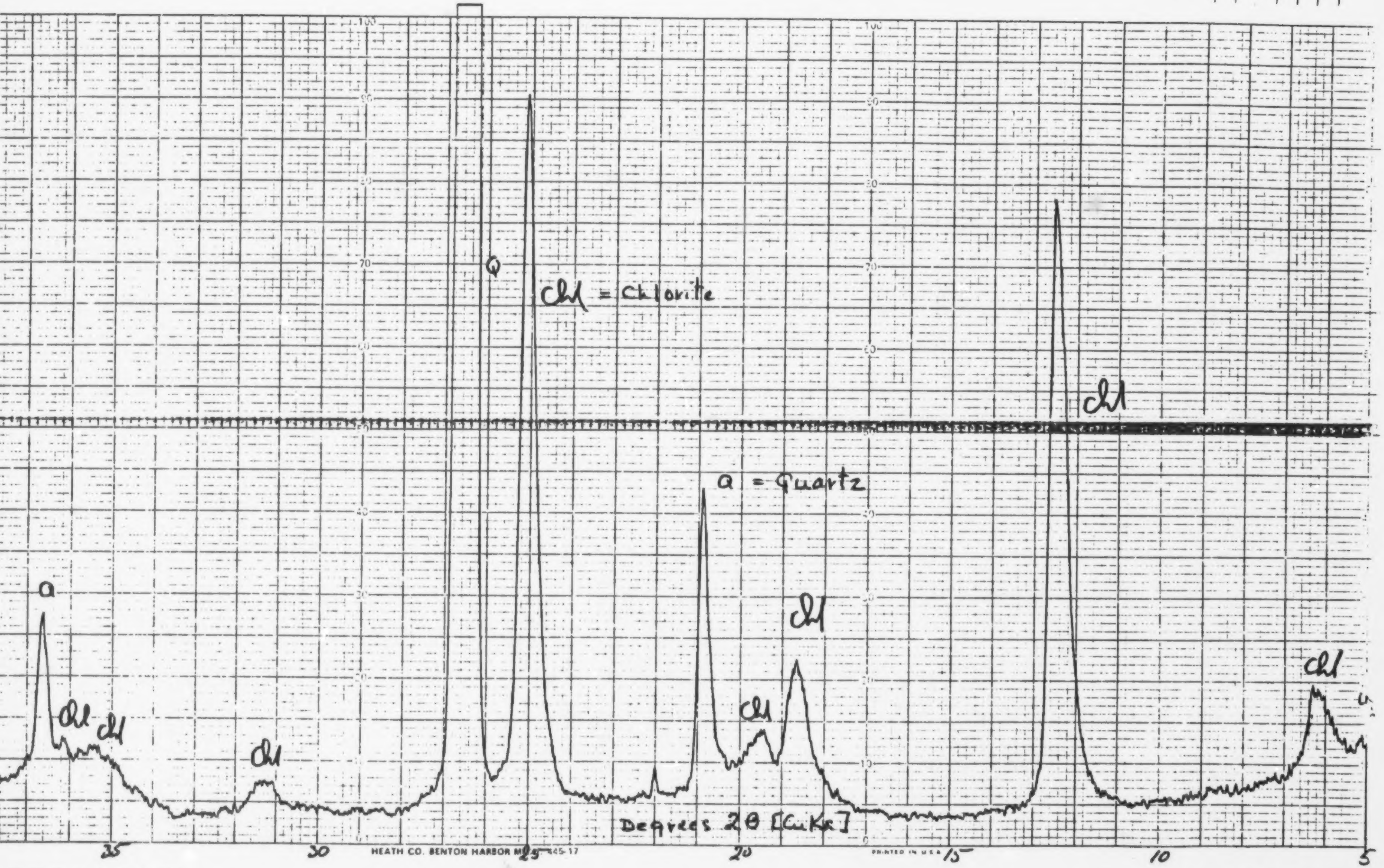

Fig. 14 XRD chart of the +44u fraction of sample #8 after 48 hour leach. Chart shows abundant quartz and some chlorite.

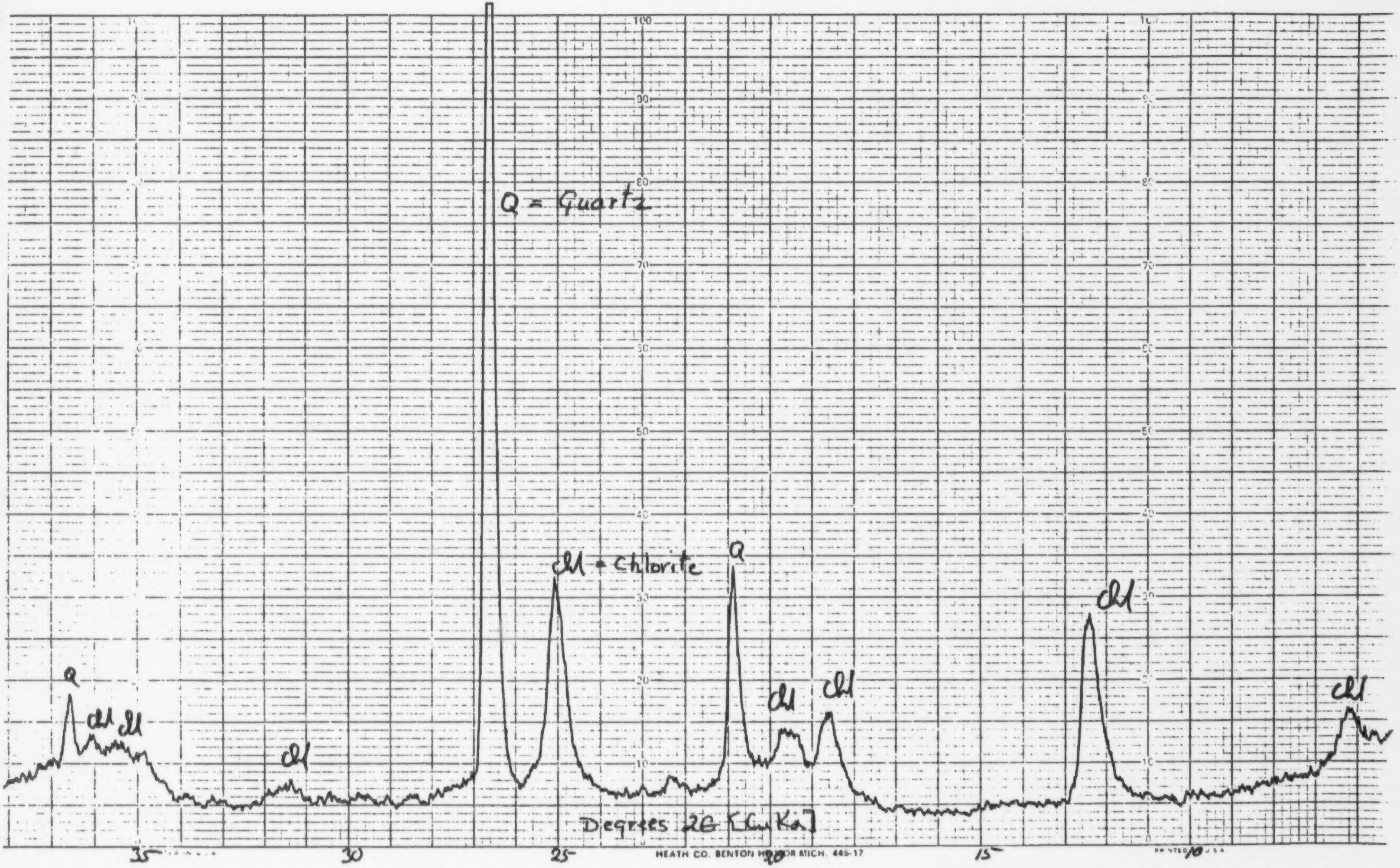

Fig. 15 XRD chart of the 44 - 5u fraction of sample #8 after 48 hour leach. Chart shows abundant quartz and some chlorite

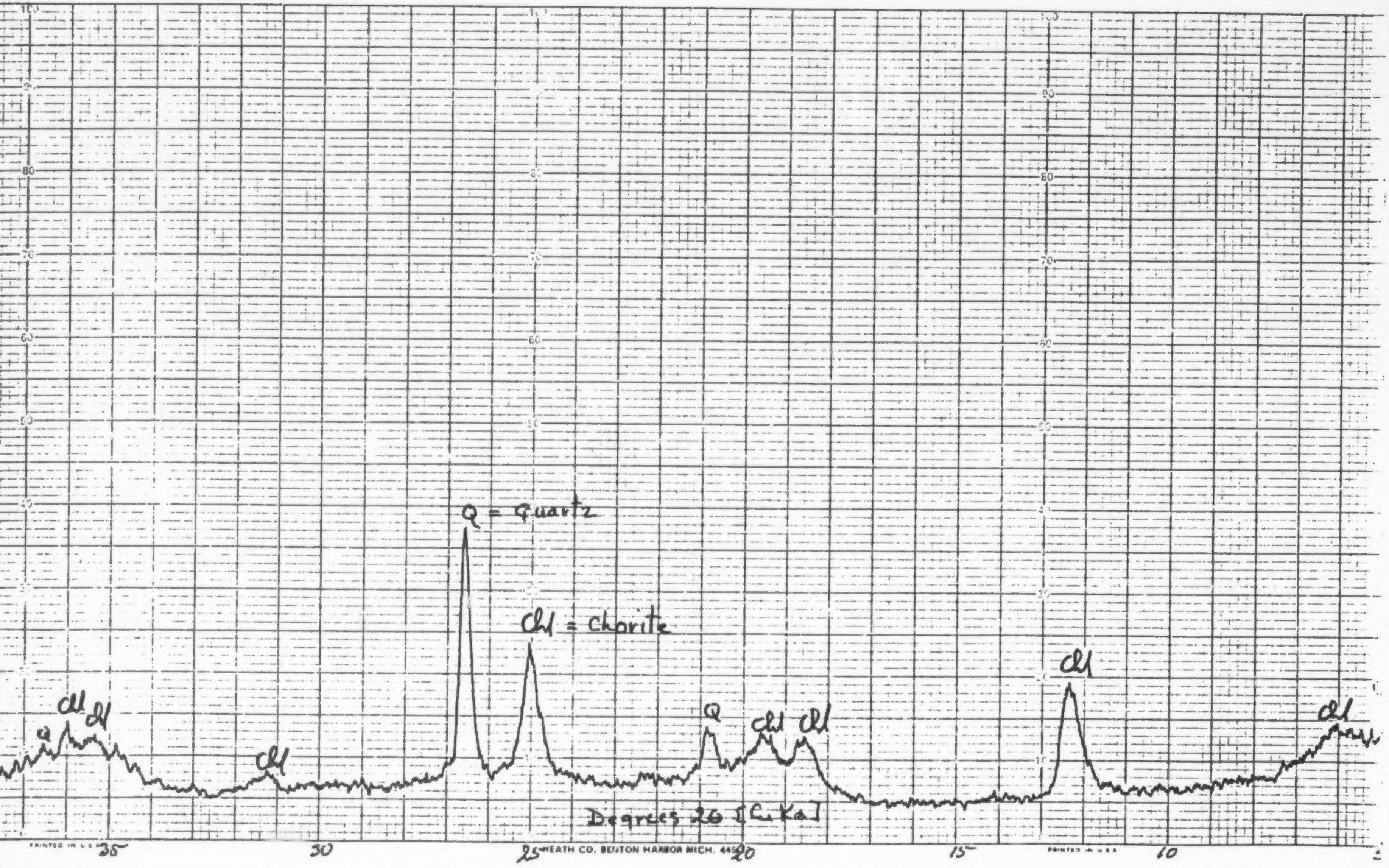

Fig. 16 XRD chart of the -5u fraction of sample #8 after 48 hour leach. Chart shows abundant quartz and chlorite. Reduced peak intensities are probably due to the fine grain size

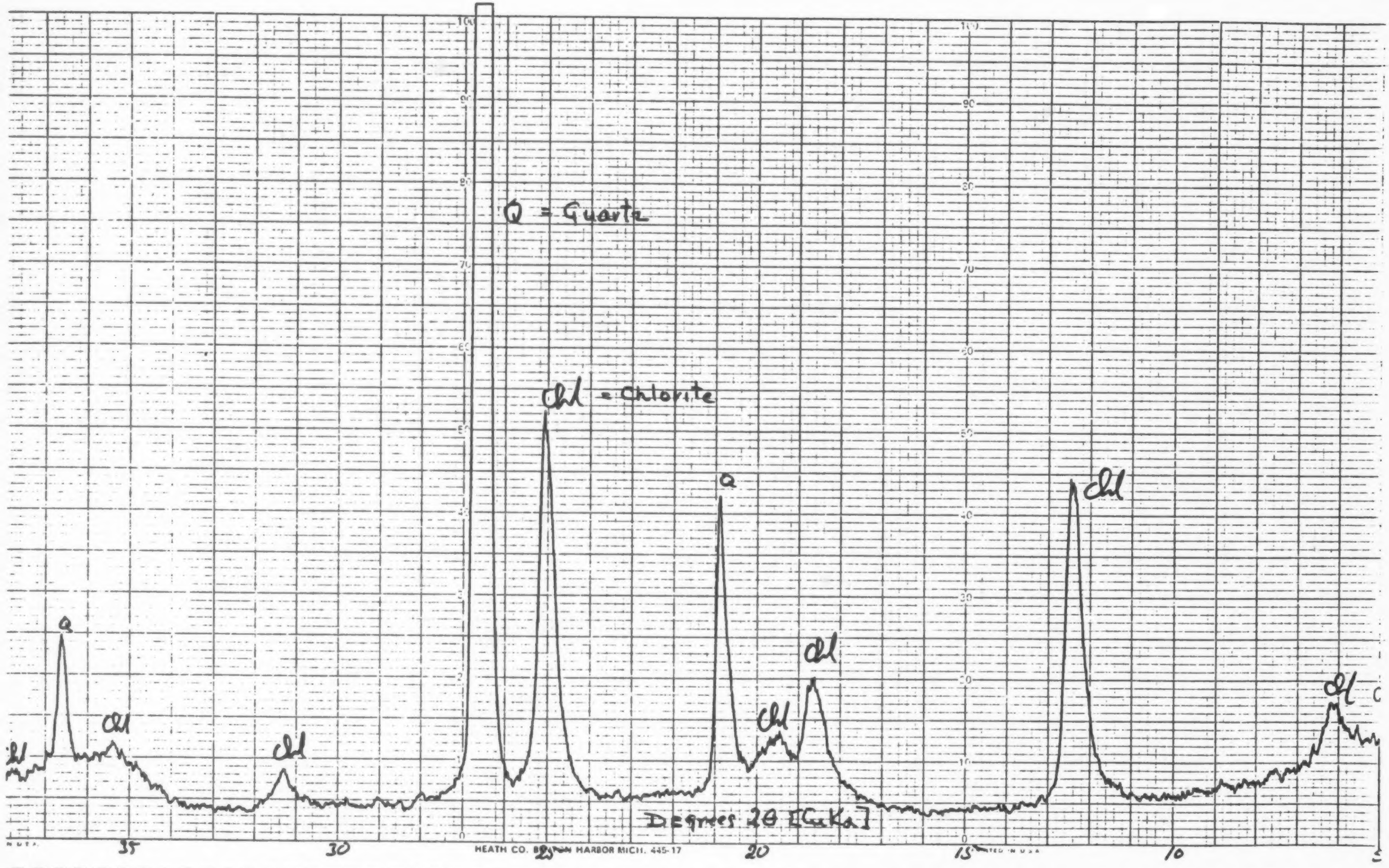

Fig. 17 XRD chart of the +44u fraction of sample #11 after 48 hour leach. Chart shows abundant quartz and some chlorite.

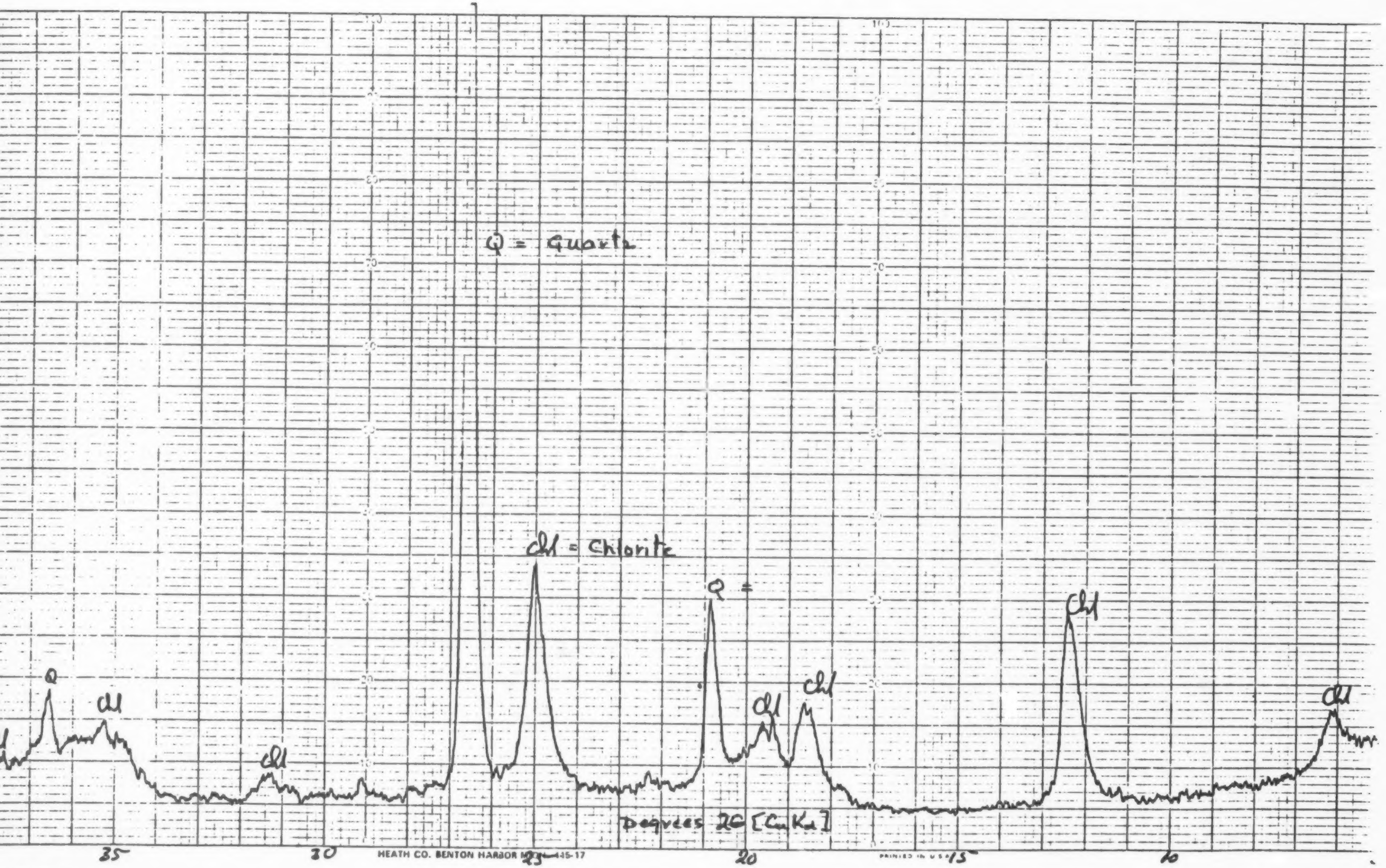

Fig. 18 XRD chart of the 44 - 5u fraction of sample #11 after 48 hour leach. Chart shows abundant quartz and some chlorite.

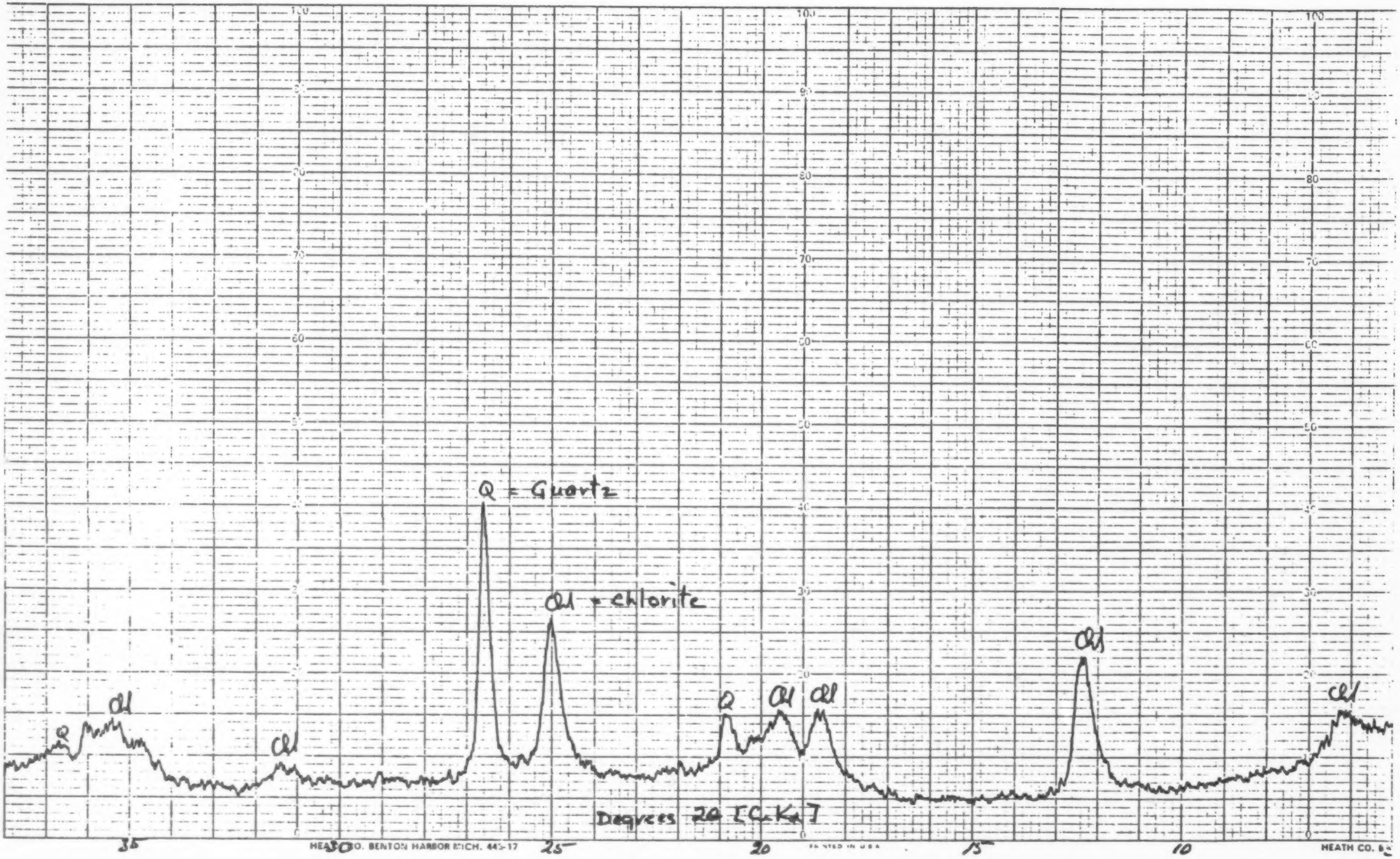

Fig. 19 XRD chart of the -5u fraction cf sample #11 after 48 hour leach, showing abundant quartz and chlorite. Reduced peak intensities are probably due to the fine grain size.

APPENDIX IV

STATEMENT OF WORK

CONTINUATION

CONTRACT SERIAL NUMBER / NUMÉRO DE SÉRIE DU CONTRAT

DSS FILE NUMBER – NUMÉRO DU DOSSIER MAS: 15SQ.23241-5-1708

Appendix "A"

STATEMENT OF WORK

Consolidation of Uranium Mill Tailings

Background:

Mineral wastes from mining and milling operations are usually hydraulically transported to a tailings impoundment area as a slurry. Because of process and pumping requirements, the amount of water used in the transport greatly exceeds the free moisture present with the solids at terminal settled density. Decant of the excess water for treatment before release (if necessary) or recycle to process, accentuates the segregation of fine particles from coarse in the tailings impoundment. The process of solids settlement combined with size segregation leads to a poorly consolidated mass with undesireable characteristics.

In the Canadian uranium ore processing industry, tailings have been traditionally deposited as "thin" slurries into impoundment basins with natural processes of sedimentation and segregation allowed to take place. Uranium tailings exhibit enhanced segregation characteristics because of the formation of fine hydrophyllic chemical precipitates in the ore treatment process. Poor consolidation and size segregation enhances hydraulic conductivity and reduces load-bearing capacity, undesireable characteristics for long-term stability and for minimizing the long-term care required.

The National Uranium Tailings Program requires a two part study of the science and practical aspects of uranium tailings consolidation. The first part is a review of the present knowledge of tailings consolidation and will apply this knowledge to the Canadian situation. A list of references to assist with this review is attached. The second part of this study will comprise field and laboratory investigations and analyses of these investigations. It is intended that these investigations will center on a specific uranium tailings impoundment in Northern Saskatchewan that will be decommissioned in the near future. Access to this specific property will be arranged by the National Uranium Tailings Program. Bidders are requested to not contact the property owner during the bidding process, (Eldor Mines).

It is proposed that the level of effort be split approximately equal between each part of the study.

Part A: Review and Examination of Fundamental Concepts

1. Where information is available, describe the physical and hydraulic characteristics that result from the deposition of uranium tailings in Canada.
2. Describe the fundamental processes involved in consolidation of uranium tailings and the relation of these processes to hydraulic conductivity, compression and load-bearing capacity for saturated and unsaturated tailings.
3. Apply the fundamental processes to the consolidation of existing uranium tailings in Canada. Three types of tailings with their corresponding impoundment system should be considered: Low grade pyritic, medium grade neutral, and high grade arsenic-nickel-clay. Include a discussion of the effects of cold weather.
4. Define the problems resulting from slime-sand segregation, and low consolidation as they relate to long-term environmental aspects of uranium tailings.
5. Describe practical methods to reduce segregation and maximize consolidation during operation of the three different typical impoundment systems. This description should include a discussion of uncertainties, costs and benefits for the long-term.

For purposes of these discussions, it should be assumed that a soil/rip-rap cover of at least 0.6 meter depth will be placed on the tailings after milling has ceased.

6. Describe practical ways to promote consolidation of existing uranium tailings deposits, including the discussion and the assumption in (5) above.

Part B: Test Program and Analyses

1. Make field measurements at, and obtain representative samples of tailings from an active tailings impoundment in Northern Saskatchewan. It is expected that the surface of this impoundment would not support mechanized equipment before mid-January 1986.
2. Conduct laboratory experiments to determine the physical characteristics of the tailings as related to consolidation and segregation.
3. Analyse field and laboratory data and relate to overall impoundment facility.
4. Propose practical methods to consolidate the tailings in situ so that maintenance of a stable cover and minimization of porewater movement is achieved in the short and long-term.

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CONTINUATION

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References:

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